Gratitude

In appreciation and gratitude
to The Custodian of the Two Holy Mosques

King Abdullah Bin Abdul Aziz Al Saud

And

H.R.H. Prince Sultan Bin Abdul Aziz Al Saud

Crown Prince, Deputy Premier, Minister of Defence
& Aviation and Inspector General

For their continuous support and gracious consideration,
the Saudi Building Code National Committee (SBCNC)
is honored to present the first issue of
the Saudi Building Code (SBC).

Saudi Building Code Requirements

201	Architectural	
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PREFACE

The Saudi Building Code (SBC) is a set of legal, administrative and technical regulations and requirements that specify the minimum standards of construction for building in order to ensure public safety and health. A Royal Decree dated 11th June 2000 order the formation of a national committee composed of representatives of Saudi universities and governmental and private sectors. In September 2001, the Council of Ministers approved the general plan of the National Committee to develop a national building code for the Kingdom of Saudi Arabia.

To choose a base code for the Saudi Building Code, a number of Codes have been studied. The National Committee has been acquainted with the results of the national researches and the international codes from the U.S.A., Canada and Australia, also, the European Code, and Arab Codes. It has also sought the opinions of specialists in relevant Saudi universities, governmental and private sectors through holding a questionnaire, a symposium and specialized workshops, in the light of which, (ICC) has been chosen to be a base code for the Saudi Building Code.

The International Code Council (ICC) grants permission to the Saudi Building Code National Committee (SBCNC) to include all or any portion of material from the ICC codes, and standards in the SBC and ICC is not responsible or liable in any way to SBCNC or to any other party or entity for any modifications or changes that SBCNC makes to such documents.

Toward expanding the participation of all the specialists in the building and construction industry in the Kingdom through the governmental and private sectors, the universities and research centers, the National Committee took its own decisions related to code content by holding specialized meetings, symposiums and workshops and by the help of experts from inside and outside of Saudi Arabia.

The technical committees and sub-committees started their work in April 2003 to develop the Saudi Building Code that adapts the base code with the social and cultural environment, the natural and climatic conditions, types of soil and properties of materials in the Kingdom.

The Saudi Building Code Structural Requirements for Steel Structures (SBC 306) were developed based on ICC code in addition to American Institute of Steel Construction Inc. (AISC). AISC grants a limited license to the Saudi Building Code National Committee (SBCNC) to utilize the following AISC publications in development of building codes and similar construction standards Specification for Structural Steel Buildings (ANSI / AISC 360-05), Seismic Provisions for Structural Steel Buildings (ANSI / AISC 341-05), Code of Standard Practice for Structural Steel Buildings and Bridges (AISC 303-05) and Specification for Safety – Related Steel Structures for Nuclear Facilities (ANSI / AISC N690-06).

The development process of SBC 306 followed the methodology approved by the Saudi Building Code National Committee. Many changes and modifications were made on AISC-LRFD, 1999, such as merging the Appendices into the main text of the Code and deleting parts or paragraphs of the Appendices and the Commentaries that are irrelevant to the Saudi Building Code. Only SI units are used through out the Code.

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SBC 306 was reviewed and significantly revised by special technical committee during the period Aug.-Sept. 2007. The committee members are:

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CHAPTER 1
GENERAL PROVISIONS

SECTION 1.1
SCOPE

The Saudi Building Code for Steel Structures referred to as SBC 306, provides minimum requirements for design and construction of Steel Structures. SBC 306 shall govern the design, fabrication, and erection of steel-framed buildings.

- Seismic design of buildings shall comply with the *AISC Seismic Provisions for Structural Steel Buildings, Seismic Provision* supplement No. 1 and with this Code.
- Single angle members shall comply with the AISC specification for *Load and Resistance Factor Design of Single-Angle Members and* with this Code.
- Hollow structural sections (HSS) shall comply with the AISC *Specification for the Design of Steel Hollow Structural Sections* and with this Code.

As used in this code, the term *structural steel* refers to the steel elements of the structural steel frame essential to the support of the required loads.

SECTION 1.2
TYPES OF CONSTRUCTION

Two basic types of construction and associated design assumptions shall be permitted under the conditions stated herein, and each will govern in a specific manner the strength of members and the types and strength of their connections.

Type FR (fully restrained), commonly designated as “rigid-frame” (continuous frame), assumes that connections have sufficient stiffness to maintain the angles between intersecting members.

Type PR (partially restrained) assumes that connections have insufficient stiffness to maintain the angles between intersecting members. When connection restraint is considered, use of Type PR construction under this code requires that the strength, stiffness and ductility characteristics of the connections be incorporated in the analysis and design. These characteristics shall be documented in the technical literature or established by analytical or experimental means.

When connection restraint is ignored, commonly designated “simple framing,” it is assumed that for the transmission of gravity loads the ends of the beams and girders are connected for shear only and are free to rotate. For “simple framing” the following requirements apply:

(1) The connections and connected members shall be adequate to resist the factored gravity loads as “simple beams.”

(2) The connections and connected members shall be adequate to resist the factored lateral loads.

(3) The connections shall have sufficient inelastic rotation capacity to avoid overload of fasteners or welds under combined factored gravity and lateral loading. The type of construction assumed in the design shall be indicated on the design documents. The design of all connections shall be consistent with the assumption.

SECTION 1.3
MATERIAL

1.3.1 Structural Steel

1.3.1.1 **ASTM Designations**. Material conforming one of the following standard specifications is approved for use under this code:

Carbon Structural Steel, ASTM A36/A36M

Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless ASTM A53/A53M, Gr. B

High-Strength Low-Alloy Structural Steel, ASTM A242/A242M

Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes, ASTM A500

Hot-Formed Welded and Seamless Carbon Steel Structural Tubing, ASTM 501

High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding, ASTM A514/A514M

High-Strength Carbon-Manganese Steel of Structural Quality, ASTM A529/A529M

Steel, Sheet and Strip, Carbon, Hot-Rolled, Structural Quality, ASTM A570/ A570M, Gr. 275, 310, and 345

High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality, ASTM A572/A572M

High-Strength Low-Alloy Structural Steel with 345 MPa Minimum Yield Point to 100 mm Thick, ASTM A588/A588M

Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance, ASTM A606

Steel, Sheet and Strip, High-Strength, Low-Alloy, Columbium or Vanadium, or Both, Hot-Rolled and Cold-Rolled, ASTM A607

Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing, ASTM A618

Carbon and High-Strength Low-Alloy Structural Steel Shapes, Plates and Bars and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges, ASTM A709/A709M

Quenched and Tempered Low-Alloy Structural Steel Plate with 485 MPa Minimum Yield Strength to 100 mm Thick, ASTM A852/A852M

High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST), ASTM A913/A913M Steel for Structural Shapes for Use in Building Framing, ASTM A992/A992M

Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M, Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling or A568/A568M, Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for, as applicable, shall constitute sufficient evidence of conformity with one of the above ASTM standards. If requested, the fabricator shall provide an affidavit stating that the structural steel furnished meets the requirements of the grade

specified.

***Note**: Materials with other international designations (e.g. JIS, EN) considered equivalent to ASTM are also approved for use under this code.*

1.3.1.2 **Unidentified Steel.** Unidentified steel, if surface conditions are acceptable according to criteria contained in ASTM A6/A6M, is permitted to be used for unimportant members or details, where the precise physical properties and weldability of the steel would not affect the strength of the structure.

1.3.1.3 **Heavy Shapes.** For ASTM A6/A6M Group 4 and 5 rolled shapes to be used as members subject to primary tensile stresses due to tension or flexure, toughness need not be specified if splices are made by bolting. If such members are spliced using complete-joint-penetration groove welds, the steel shall be specified in the contract documents to be supplied with Charpy V-notch (CVN) impact testing in accordance with ASTM A6/A6M, Supplementary Requirement S5. The impact test shall meet a minimum average value of 27 J absorbed energy at +21°C and shall be conducted in accordance with ASTM A673/A673M, with the following exceptions:

1. The center longitudinal axis of the specimens shall be located as near as practical to midway between the inner flange surface and the center of the flange thickness at the intersection with the web mid-thickness.
2. Tests shall be conducted by the producer on material selected from a location representing the top of each ingot or part of an ingot used to produce the product represented by these tests.

For plates exceeding 50 mm thick used for built-up cross-sections with bolted splices and subject to primary tensile stresses due to tension or flexure, material toughness need not be specified. If such cross-sections are spliced using complete-joint-penetration welds, the steel shall be specified in the contract documents to be supplied with Charpy V-notch testing in accordance with ASTM A6/A6M, Supplementary Requirement S5. The impact test shall be conducted by the producer in accordance with ASTM A673/A673M, Frequency P, and shall meet a minimum average value of 27 J absorbed energy at +21°C.

The above supplementary requirements also apply when complete-joint-penetration welded joints through the thickness of ASTM A6/A6M Group 4 and 5 shapes and built-up cross sections with thickness exceeding 50 mm are used in connections subjected to primary tensile stress due to tension or flexure of such members. The requirements need not apply to ASTM A6/A6M Group 4 and 5 shapes and built-up members with thickness exceeding 50 mm to which members other than ASTM A6/A6M Group 4 and 5 shapes and built-up members are connected by complete-joint- penetration welded joints through the thickness of the thinner material to the face of the heavy material.

Additional requirements for joints in heavy rolled and built-up members are given in Sections 10.1.5, 10.2.8 and 13.2.2.

***Note**: Materials with other international designations (e.g. JIS, EN) considered equivalent to ASTM are also approved for use under this code.*

1.3.2 **Steel Castings and Forgings.** Cast steel shall conform to one of the following standard specifications:

Steel Castings, Carbon, for General Application, ASTM A27/A27M, Gr. 450-240

Steel Castings, High Strength, for Structural Purposes, ASTM A148/148M Gr.

550-345

Steel forgings shall conform to the following standard specification:

Steel Forgings Carbon and Alloy, for General Industrial Use, ASTM A668/ A668M

Certified test reports shall constitute sufficient evidence of conformity with standards.

Note*: Materials with other international designations (e.g. JIS, EN) considered equivalent to ASTM are also approved for use under this code.*

1.3.3 **Bolts, Washers, and Nuts.** Steel bolts, washers, and nuts shall conform to one of the following standard specifications:

Carbon and Alloy Steel Nuts for Bolts for High-Pressure or High-Temperature Service, or Both, ASTM A194/A194M

Carbon Steel Bolts and Studs, 410 MPa Tensile Strength, ASTM A307

Structural Bolts, Steel, Heat Treated, 830/720 MPa Minimum Tensile Strength, ASTM A325

High-Strength Bolts for Structural Steel Joints [Metric], ASTM A325M Quenched and Tempered Steel Bolts and Studs, ASTM A449

Heat-Treated Steel Structural Bolts, 1030 MPa Minimum Tensile Strength, ASTM A490

High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric], ASTM A490M

Carbon and Alloy Steel Nuts, ASTM A563

Carbon and Alloy Steel Nuts [Metric], ASTM A563M

Hardened Steel Washers, ASTM F436

Hardened Steel Washers [Metric], ASTM F436M

Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners, ASTM F959

Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners [Metric], ASTM F959M

“Twist Off” Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 830/720 MPa Minimum Tensile Strength, ASTM F1852

ASTM A449 bolts are permitted to be used only in connections requiring bolt diameters greater than 38 mm and shall not be used in slip-critical connections. Manufacturer’s certification shall constitute sufficient evidence of conformity with the standards.

Note*: Materials with other international designations (e.g. JIS, EN) considered equivalent to ASTM are also approved for use under this code.*

1.3.4 **Anchor Rods and Threaded Rods.** Anchor rods and threaded rod steel shall conform to one of the following standard specifications:

Carbon Structural Steel, ASTM A36/A36M

Alloy Steel and Stainless Steel Bolting Materials for High-Temperature Service, ASTM A193/A193M

Quenched and Tempered Alloy Steel Bolts, Studs and Other Externally Threaded Fasteners, ASTM A354

High-Strength Low-Alloy Columbium-Vanadium Structural Steel, ASTM A572/A572M

High-Strength Low-Alloy Structural Steel with 345 MPa Minimum

Yield Point to 100 mm Thick, ASTM A588/A588M

Anchor Bolts, Steel, 250, 380, 720 MPa - Yield Strength, ASTM F1554

Threads on anchor rods and threaded rods shall conform to the Unified Standard Series of ASME B18.2.6 and shall have Class 2A tolerances.

Steel bolts conforming to other provisions of Section 1.3.3 are permitted as anchor rods. A449 material is acceptable for high-strength anchor rods and threaded rods of any diameter.

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

***Note**: Materials with other international designations (e.g. JIS, EN) considered equivalent to ASTM are also approved for use under this code.*

1.3.5 **Filler Metal and Flux for Welding.** Filler metals and fluxes shall conform to one of the following specifications of the American Welding Society:

Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding, AWS A5.1

Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding, AWS A5.5

Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding, AWS A5.17/A5.17M

Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding, AWS A5.18

Specification for Carbon Steel Electrodes for Flux Cored Arc Welding, AWS A5.20

Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding, AWS A5.23/A5.23M

Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding, AWS A5.25/A5.25M

Specification for Carbon and Low-Alloy Steel Electrodes for Electrogas Welding, AWS A5.26/A5.26M

Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding, AWS A5.28

Specification for Low-Alloy Steel Electrodes for Flux Cored Arc Welding, AWS A5.29

Specification for Welding Shielding Gases, AWS A5.32/A5.32M Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

Filler metals and fluxes that are suitable for the intended application shall be selected.

1.3.6 **Stud Shear Connectors.** Steel stud shear connectors shall conform to the requirements of *Structural Welding Code—Steel*, AWS D1.1.

Manufacturer's certification shall constitute sufficient evidence of conformity with the specifications.

SECTION 1.4
LOADS AND LOAD COMBINATIONS

The nominal loads and factored load combinations shall be as stipulated by SBC 301.

SECTION 1.5
DESIGN BASIS

1.5.1 **Required Strength at Factored Loads.** The required strength of structural members and connections shall be determined by structural analysis for the appropriate factored load combinations as stipulated in Section 1.4.

Design by either elastic or plastic analysis is permitted, except that design by plastic analysis is permitted only for steels with specified minimum yield stresses not exceeding 450 MPa and is subject to provisions of Sections 2.5.2, 3.1.1, 3.2.1, 3.2.2, 5.1.2, 6.1.3, 8.1, and 9.1.

Beams and girders composed of compact sections, as defined in Section 2.5.1, and satisfying the unbraced length requirements of Section 6.1.3 (including composite members) which are continuous over supports or are rigidly framed to columns may be proportioned for nine-tenths of the negative moments produced by the factored gravity loading at points of support, provided that the maximum positive moment is increased by one-tenth of the average negative moments. This reduction is not permitted for hybrid beams, members of A514/A514M steel, or moments produced by loading on cantilevers. If the negative moment is resisted by a column rigidly framed to the beam or girder, the one-tenth reduction may be used in proportioning the column for combined axial force and flexure, provided that the axial force does not exceed ϕ_c times $0.15A_gF_y$,

Where:

A_g = gross area, mm^2
F_y = specified minimum yield stress, MPa
ϕ_c = resistance factor for compression

1.5.2 **Limit States.** LRFD is a method of proportioning structures so that no applicable limit state is exceeded when the structure is subjected to all appropriate factored load combinations.

Strength limit states are related to safety and concern maximum load carrying capacity. Serviceability limit states are related to performance under normal service conditions. The term "resistance" includes both strength limit states and serviceability limit states.

1.5.3 **Design for Strength.** The required strength shall be determined for each applicable load combination as stipulated in Section 1.4.

The design strength of each structural component or assemblage shall equal or exceed the required strength based on the factored loads. The design strength ϕR_n for each applicable limit state is calculated as the nominal strength R_n multiplied by a resistance factor. Nominal strengths R_n and resistance factors are given in Chapters 4 through 11.

1.5.4 **Design for Serviceability and Other Considerations.** The overall structure and the individual members, connections, and connectors shall be checked for serviceability. Provisions for design for serviceability are given in Chapter 12.

SECTION 1.6
DESIGN DOCUMENTS

The design drawings shall show a complete design with sizes, sections, and relative locations of all members. Floor levels, column centers and offsets shall be dimensioned. Drawings shall be drawn to a scale large enough to show the information clearly.

Design documents shall indicate the type or types of construction as defined in Section 1.2 and include the required strengths (moments and forces) if necessary for preparation of shop drawings.

Where joints are to be assembled with high-strength bolts, the design documents shall indicate the connection type (i.e., snug-tightened, pretensioned, or slip-critical).

Camber of trusses, beams, and girders, if required, shall be specified in the design documents.

The requirements for stiffeners and bracing shall be shown in the design documents.

Welding and inspection symbols used on design and shop drawings shall be the American Welding Society symbols. Welding symbols for special requirements not covered by AWS are permitted to be used provided complete explanations thereof are shown in the design documents.

Weld lengths called for in the design documents and on the shop drawings shall be the net effective lengths.

CHAPTER 2
DESIGN REQUIREMENTS

This chapter contains provisions, which are common to this code as a whole.

SECTION 2.1
GROSS AREA

The gross area A_g of a member at any point is the sum of the products of the thickness and the gross width of each element measured normal to the axis of the member. For angles, the gross width is the sum of the widths of the legs less the thickness.

SECTION 2.2
NET AREA

Critical net area is based on net width and load transfer at a particular chain.

The net area A_n of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as 2 mm greater than the nominal dimension of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section 10.3.2, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/4g$

Where:

s = longitudinal center-to-center spacing (pitch) of any two consecutive holes, mm.

g = transverse center-to-center spacing (gage) between fastener gage lines, mm.

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

In determining the net area across plug or slot welds, the filler metal shall not be considered as adding to the net area.

SECTION 2.3
EFFECTIVE AREA OF TENSION MEMBERS

The effective area of tension members shall be determined as follows:

(1) When tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds, the effective area A_e is equal to the net area A_n.

(2) When the tension load is transmitted by fasteners or welds through some but not all of the cross-sectional elements of the member, the effective area A_e shall be computed as follows:

(a) When the tension load is transmitted only by fasteners

$$A_e = A_n U \qquad \textbf{(2.3-1)}$$

Where:
U = reduction coefficient
$= 1-(\bar{x}/l) \leq 0.9$
$\bar{x}$ = connection eccentricity, mm
l = length of the connection in the direction of loading, mm

(b) When the tension load is transmitted only by longitudinal welds to other than a plate member or by longitudinal welds in combination with transverse welds

$$A_e = A_g U \qquad \textbf{(2.3-2)}$$

Where:
U = $1-(\bar{x}/l) \leq 0.9$
A_g = gross area of member, mm^2

(c) When the tension load is transmitted only by transverse welds

$$A_e = AU \qquad \textbf{(2.3-3)}$$

Where:
A = area of directly connected elements, mm^2
U = 1.0

(d) When the tension load is transmitted to a plate only by longitudinal welds along both edges at the end of the plate

$$A_e = A_g U \qquad \textbf{(2.3-4)}$$

Where:
For $l \geq 2w$ . . . . $U = 1.00$
For $2w > l \geq 1.5w$ . . .. $U = 0.87$
For $1.5w > l \geq w$ . . . . $U = 0.75$
Where:
l = length of weld, mm
w = plate width (distance between welds), mm

The reduction coefficient U is applied to the net area A_n of bolted members and to the gross area A_g of welded members. As the length of connection l is increased, the shear lag effect is diminished. This concept is expressed empirically by the equation for U.

For any given profile and connected eleme*nts* $\bar{x}$ is a fixed geometric property. It is illustrated as the distance from the connection plane, or face of the member, to the centroid of the member section resisting the connection force. See Figure 2.3-1. The length l is dependent upon the number of fasteners or equivalent length of weld required to develop the given tensile force, and this in turn is dependent upon the mechanical properties of the member and the capacity of the fasteners or weld used. The length l is illustrated as the distance, parallel to the line of force, between the first and last fasteners in a line for bolted connections. The number of bolts in a line, for the purpose of the determination of l, is determined by the line with the maximum number of bolts in the connection. For staggered bolts, the out-to-out dimension is used for l. See Figure 2.3-2. If all lines have only one bolt, it is probably conservative to use A_e equal to the net area of the connected element. For welded connections, l is the length of the weld parallel to the line of force. For combinations of longitudinal and transverse welds (see Figure 2.3-3), l is the

length of longitudinal weld because the transverse weld has little or no effect on the shear lag problem, i.e., it does little to get the load into the unattached portions of the member.

For bolted or riveted connections the following values of U may be used:

(a) W, M, or S shapes with flange widths not less than two-thirds the depth, and structural tees cut from these shapes, provided the connection is to the flanges and has no fewer than three fasteners per line in the direction of stress, $U = 0.90$.

(b) W, M, or S shapes not meeting the conditions of subparagraph a, structural tees cut from these shapes, and all other shapes including built-up cross sections, provided the connection has no fewer than three fasteners per line in the direction of stress, $U = 0.85$.

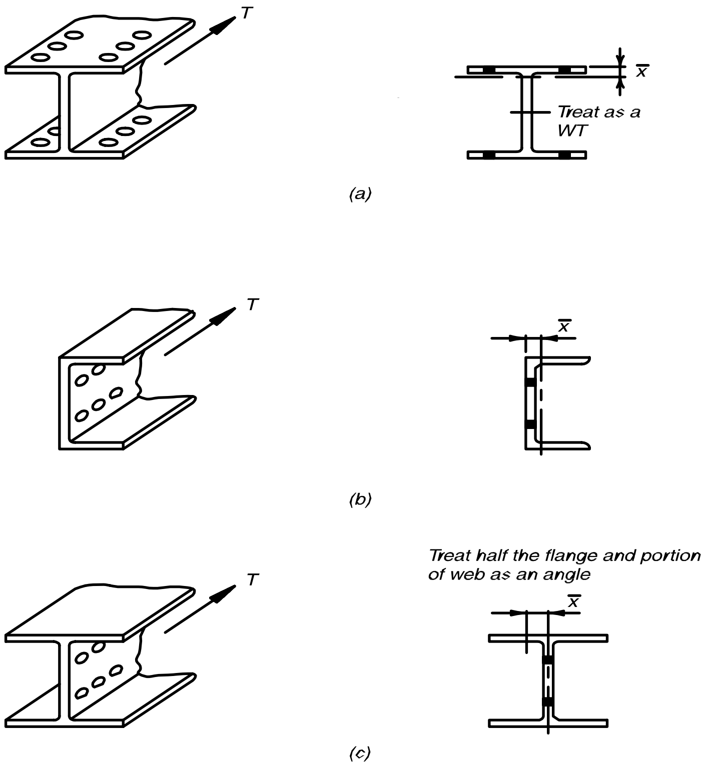


Figure 2.3-1 Determination of $\bar{x}$ for U.

(c) All members having only two fasteners per line in the direction of stress, $U = 0.75$.

When a tension load is transmitted by fillet welds to some but not all elements of a cross section, the weld strength will control.

Larger values of U are permitted to be used when justified by tests or other rational criteria.

For effective area of connecting elements, see Section 10.5.2.

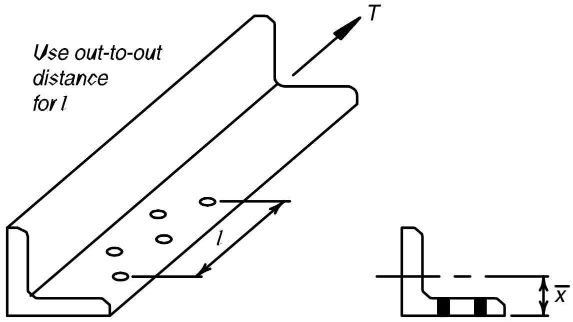


Figure 2.3-2. Staggered holes.

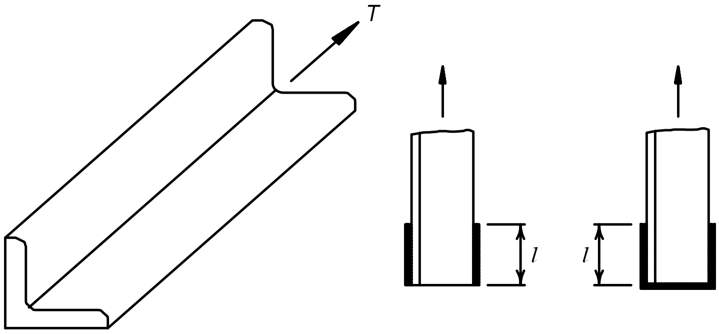


Figure 2.3-3. Longitudinal and transverse welds.

SECTION 2.4
STABILITY

General stability shall be provided for the structure as a whole and for each of its elements.
Consideration shall be given to the significant effects of the loads on the deflected shape of the structure and its individual elements.

SECTION 2.5
LOCAL BUCKLING

2.5.1 **Classification of Steel Sections.** Steel sections are classified as compact, non-compact, or slender-element sections. For a section to qualify as compact, its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios λ_p from Table 2.5-1. If the width-thickness ratio of one or more compression elements exceeds λ_p, but does not exceed λ_r, the section is non-compact. If the width-thickness ratio of any element exceeds λ_r from Table 2.5-1, the section is referred to as a slender-element compression section.

For un-stiffened elements which are supported along only one edge parallel to the direction of the compression force, the width shall be taken as follows:

(a) For flanges of I-shaped members and tees, the width b is half the full-flange width, b_f.
(b) For legs of angles and flanges of channels and zees, the width b is the full nominal dimension.
(c) For plates, the width b is the distance from the free edge to the first row of fasteners or line of welds.

(d) For stems of tees, d is taken as the full nominal depth.

For stiffened elements which are supported along two edges parallel to the direction of the compression force, the width shall be taken as follows:

(a) For webs of rolled or formed sections, h is the clear distance between flanges less the fillet or corner radius at each flange; h_c is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.

(b) For webs of built-up sections, h is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, and h_c is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used; h_p is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.

(c) For flange or diaphragm plates in built-up sections, the width b is the distance between adjacent lines of fasteners or lines of welds.

(d) For flanges of rectangular hollow structural sections, the width b is the clear distance between webs less the inside corner radius on each side. If the corner radius is not known, the width may be taken as the total section width minus three times the thickness. The thickness t shall be taken as the design wall thickness. When the design wall thickness is not known, it is permitted to be taken as 0.93 times the nominal wall thickness.

The limiting width-thickness ratio for: a) the design of webs in combined flexure and axial compression and, b) the design of members containing slender compression elements are as follows:

- For members with unequal flanges and with webs in combined flexural and axial compression, λ_r for the limit state of web local buckling is

$$\lambda_r = 1.49\sqrt{\frac{E}{F_y}}\left[1 + 2.83\left(\frac{h}{h_c}\right)\left(1 - \frac{P_u}{\phi_b P_y}\right)\right] \quad \textbf{(2.5-1)}$$

$$\frac{3}{4} \le \frac{h}{h_c} \le \frac{3}{2}$$

- For members with unequal flanges with webs subjected to flexure only, λ_r for the limit state of web local buckling is

$$\lambda_r = 1.49\sqrt{\frac{E}{F_y}}\left[1 + 2.83\left(\frac{h}{h_c}\right)\right] \quad \textbf{(2.5-2)}$$

$$\frac{3}{4} \le \frac{h}{h_c} \le \frac{3}{2}$$

where λ_r, h, and h_c are as defined in Section 2.5.1.

These substitutions shall be made in Sections 6 and 7 when applied to members with unequal flanges. If the compression flange is larger than the tension flange, λ_r shall be determined using Equation 2.5-1, 2.5-2, or Table 2.5-1.

For tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

2.5.2 **Design by Plastic Analysis.** Design by plastic analysis is permitted, as limited in Section 1.5.1, when flanges subject to compression involving hinge rotation

and all webs have a width-thickness ratio less than or equal to the limiting λ_p from Table 2.5-1. For circular hollow sections see Footnote d of Table 2.5-1.

2.5.3 **Slender-Element Compression Sections.** For the flexural design of I-shaped sections, channels and rectangular or circular sections with slender flange elements, see section 6.1. For other shapes in flexure or members in axial compression that have slender compression elements, see Section 2.5.3. For plate girders with slender web elements, Section 7.

Axially loaded members containing elements subject to compression which have a width-thickness ratio in excess of the applicable λ_r as stipulated in Section 2.5.1 shall be proportioned according to this section. Flexural members with slender compression elements shall be designed in accordance with Sections 6 and 7. Flexural members with proportions not covered by Section 6.1 shall be designed in accordance with this Section.

The limiting width-thickness ratio for: the design of members containing slender compression elements are given below in Sections 2.5.3.1 to 2.5.3.4.

2.5.3.1 **Un-stiffened Compression Elements.** The design strength of un-stiffened compression elements whose width-thickness ratio exceeds the applicable limit λ_r as stipulated in Section 2.5.1 shall be subject to a reduction factor Q_s. The value of Q_s shall be determined by Equations 2.5-3 through 2.5-10, as applicable. When such elements comprise the compression flange of a flexural member, the design flexural strength, in MPa, shall be computed using $\phi_b F_y Q_s$, where $\phi_b = 0.90$. The design strength of axially loaded compression members shall be modified by the appropriate reduction factor Q, as provided in Section 2.5.3.4.

(a) For single angles:

when $0.45\sqrt{E/F_y} < b/t < 0.91\sqrt{E/F_y}$:

$$Q_s = 1.340 - 0.76(b/t)\sqrt{F_y/E} \qquad \textbf{(2.5-3)}$$

when $b/t \geq 0.91\sqrt{E/F_y}$:

$$Q_s = 0.53E/\left[F_y(b/t)^2\right] \qquad \textbf{(2.5-4)}$$

(b) For flanges, angles, and plates projecting from rolled beams or columns or other compression members:

when $0.56\sqrt{E/F_y} < b/t < 1.03\sqrt{E/F_y}$:

$$Q_s = 1.415 - 0.74(b/t)\sqrt{F_y/E} \qquad \textbf{(2.5-5)}$$

when $b/t \geq 1.03\sqrt{E/F_y}$:

$$Q_s = 0.69E/\left[F_y(b/t)^2\right] \qquad \textbf{(2.5-6)}$$

(c) For flanges, angles and plates projecting from built-up columns or other compression members:

when $0.64\sqrt{E/(F_y/k_c)} < b/t < 1.17\sqrt{E/(F_y/k_c)}$:

$$Q_s = 1.415 - 0.65(b/t)\sqrt{(F_y / k_c E)} \qquad \textbf{(2.5-7)}$$

when $b/t \geq 1.17\sqrt{E/(F_y / k_c)}$:

$$Q_s = 0.90 E k_c / \left[F_y (b/t)^2\right] \qquad \textbf{(2.5-8)}$$

The coefficient, k_c, shall be computed as follows:

(a) For I-shaped sections:

$$k_c = \frac{4}{\sqrt{h/t_w}},\ 0.35 \leq k_c \leq 0.763$$

where:

h = depth of web, mm

t_w = thickness of web, mm

(b) For other sections:

$k_c = 0.763$

(c) For stems of tees:

when $0.75\sqrt{E/F_y} < d/t < 1.03\sqrt{E/F_y}$:

$$Q_s = 1.908 - 1.22(d/t)\sqrt{F_y / E} \qquad \textbf{(2.5-9)}$$

when $d/t \geq 1.03\sqrt{E/F_y}$:

$$Q_s = 0.69 E / \left[F_y (d/t)^2\right] \qquad \textbf{(2.5-10)}$$

where:

d = width of un-stiffened compression element as defined in Section 2.5.1, mm

t = thickness of un-stiffened element, mm

2.5.3.2 **Stiffened Compression Elements.** When the width-thickness ratio of uniformly compressed stiffened elements (except perforated cover plates) exceeds the limit λ_r stipulated in Section 2.5.1, a reduced effective width b_e shall be used in computing the design properties of the section containing the element.

(a) For flanges of square and rectangular sections of uniform thickness:

when $\frac{b}{t} \geq 1.40\sqrt{\frac{E}{f}}$:

$$b_e = 1.91t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \qquad \textbf{(2.5-11)}$$

otherwise $b_e = b$.

(b) For other uniformly compressed elements:

when $\frac{b}{t} \geq 1.49\sqrt{\frac{E}{f}}$:

$$b_e = 1.91t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \qquad \textbf{(2.5-12)}$$

otherwise $b_e = b$.

where:

b = actual width of a stiffened compression element, as defined in Section 2.5.1, mm

b_e = reduced effective width, mm

t = element thickness, mm

f = computed elastic compressive stress in the stiffened elements, based on the design properties as specified in Section 2.5.3.3, MPa. If un-stiffened elements are included in the total cross section, f for the stiffened element must be such that the maximum compressive stress in the un-stiffened element does not exceed $\phi_c F_{cr}$ as defined in Section 2.5.3.4 with $Q = Q_s$ and $\phi_c = 0.85$, or $\phi_b F_y Q_s$ with $\phi_b = 0.90$, as applicable.

(c) For axially loaded circular sections with diameter-to-thickness ratio D/t greater than $0.11E/F_y$ but less than $0.45E / F_y$

$$Q = Q_a = \frac{0.038E}{F_y(D/t)} + \frac{2}{3} \qquad \textbf{(2.5-13)}$$

where:

D = outside diameter, mm

t = wall thickness, mm

2.5.3.3 **Design Properties.** Properties of sections shall be determined using the full cross section, except as follows:

In computing the moment of inertia and elastic section modulus of flexural members, the effective width of uniformly compressed stiffened elements b_e, as determined in Section 2.5.3.2, shall be used in determining effective cross-sectional properties.

For unstiffened elements of the cross section, Q_s is determined from Section 2.5.3.1. For stiffened elements of the cross section

$$Q_a = \frac{\text{effective area}}{\text{actual area}} \qquad \textbf{(2.5-14)}$$

where the effective area is equal to the summation of the effective areas of the cross section.

2.5.3.4 **Design Strength.** For axially loaded compression members the gross cross-sectional area and the radius of gyration r shall be computed on the basis of the actual cross section. The critical stress F_{cr} shall be determined as follows:

(a) For $\lambda_c \sqrt{Q} \leq 1.5$:

$$F_{cr} = Q(0.658^{Q\lambda_c^2})F_y \qquad \textbf{(2.5-15)}$$

(b) For $\lambda_c \sqrt{Q} > 1.5$:

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right]F_y \qquad \textbf{(2.5-16)}$$

where

$$Q = Q_s Q_a \qquad \textbf{(2.5-17)}$$

Cross sections comprised of only un-stiffened elements, $Q = Q_s$, ($Q_a = 1.0$),

Cross sections comprised of only stiffened elements, $Q = Q_a$, ($Q_s = 1.0$),

Cross sections comprised of both stiffened and un-stiffened elements, $Q = Q_s Q_a$.

TABLE 2.5-1
Limiting Width-Thickness Ratios for Compression Elements

Description of Element		**Width-Thickness Ratio**	**Limiting Width-Thickness Ratios**	
			λ_p **(compact)**	λ_r **(non-compact)**
Unstiffened Elements	Flanges of I-shaped rolled beams and channels in flexure	b/t	$0.38\sqrt{E/F_y}$ [c]	$0.83\sqrt{E/F_L}$ [e]
	Flanges of I-shaped hybrid or welded beams in flexure	b/t	$0.38\sqrt{E/F_{yf}}$	$0.95\sqrt{E/(F_L/k_c)}$ [e], [f]
	Flanges projecting from built-up compression members	b/t	NA	$0.64\sqrt{E/(F_y/k_c)}$ [f]
	Flanges of I-shaped sections in pure compression, plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact; flanges of channels in pure compression	b/t	NA	$0.56\sqrt{E/F_y}$
	Legs of single angle struts; legs of double angle struts with separators; un-stiffened elements, i.e., supported along one edge	b/t	NA	$0.45\sqrt{E/F_y}$
	Stems of tees	d/t	NA	$0.75\sqrt{E/F_y}$

PROPORTIONS OF BEAMS AND GIRDERS

TABLE 2.5-1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

<table>
<tr><th colspan="2" rowspan="2">Description of Element</th><th rowspan="2">Width-Thick-ness Ratio</th><th colspan="2">Limiting Width-Thickness Ratios</th></tr>
<tr><th>λ_p
(compact)</th><th>λ_r
(non-compact)</th></tr>
<tr><td rowspan="4">Stiffened Elements</td><td>Flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds

for uniform compression

for plastic analysis</td><td>b/t</td><td>

$1.12\sqrt{E/F_y}$

$0.939\sqrt{E/F_y}$</td><td>

$1.40\sqrt{E/F_y}$</td></tr>
<tr><td>Unsupported width of cover plates perforated with a succession of access holes [b]</td><td>b/t</td><td>NA</td><td>$1.86\sqrt{E/F_y}$</td></tr>
<tr><td>Webs in flexural compression [a]</td><td>h/t_w</td><td>$3.76\sqrt{E/F_y}$ [c], [g]</td><td>$5.70\sqrt{E/F_y}$ [h]</td></tr>
<tr><td>Webs in combined flexural and axial compression</td><td>h/t_w</td><td>for $P_u/\phi_b P_y \le 0.125$ [c], [g]
$3.76\sqrt{\dfrac{E}{F_y}}\left(1-\dfrac{2.75P_u}{\phi_b P_y}\right)$

for $P_u/\phi_b P_y > 0.125$ [c], [g]
$1.12\sqrt{\dfrac{E}{F_y}}\left(2.33-\dfrac{P_u}{\phi_b P_y}\right)$
$\ge 1.49\sqrt{\dfrac{E}{F_y}}$</td><td>[h]
$5.70\sqrt{\dfrac{E}{F_y}}\left(1-0.74\dfrac{P_u}{\phi_b P_y}\right)$</td></tr>
</table>

PROPORTIONS OF BEAMS AND GIRDERS

TABLE 2.5-1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

	Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios λ_p (compact)	Limiting Width-Thickness Ratios λ_r (non-compact)
	All other uniformly compressed stiffened elements, i.e., supported along two edges	b/t h/t_w	NA	$1.49\sqrt{E/F_y}$
	Circular hollow sections In axial compression In flexure	D/t	[d] NA $0.07E/F_y$	 $0.11E/F_y$ $0.31E/F_y$

[a] For hybrid beams, use the yield strength of the flange F_{yf} instead of F_y.
[b] Assumes net area of plate at widest hole.
[c] Assumes an inelastic ductility ratio (ratio of strain at fracture to strain at yield) of 3. When the seismic response modification factor R is taken greater than 3, a greater rotation capacity may be required.
[d] For plastic design use $0.045E/F_y$.
[e] F_L = smaller of $(F_{yf} - F_r)$ or F_{yw}, (MPa)
F_r = compressive residual stress in flange
= 69 MPa for rolled shapes
= 114 MPa for welded shapes
[f] $k_c = \dfrac{4}{\sqrt{h/t_w}}$ and $0.35 \le k_c \le 0.763$
[g] For members with unequal flanges, use h_p of h when comparing to λ_p.
[h] For members with unequal flanges, see Section 2.5.

SECTION 2.6
BRACING AT SUPPORTS

At points of support for beams, girders and trusses, restraint against rotation about their longitudinal axis shall be provided.

SECTION 2.7
LIMITING SLENDERNESS RATIOS

For members in which the design is based on compression, the slenderness ratio Kl/r preferably should not exceed 200.
For members in which the design is based on tension, the slenderness ratio l/r preferably should not exceed 300. The above limitation does not apply to rods in tension. Members in which the design is dictated by tension loading, but which may be subject to some compression under other load conditions, need not satisfy the compression slenderness limit.

SECTION 2.8
SIMPLE SPANS

Beams, girders and trusses designed on the basis of simple spans shall have an effective length equal to the distance between centers of gravity of the members to which they deliver their end reactions.

SECTION 2.9
END RESTRAINT

Beams, girders, and trusses designed on the assumptions of full or partial end restraint, as well as the sections of the members to which they connect, shall have design strengths, as prescribed in Chapters 4 through 11, equal to or exceeding the effect of factored forces and moments except that some inelastic but self-limiting deformation of a part of the connection is permitted.

SECTION 2.10
PROPORTIONS OF BEAMS AND GIRDERS

When rolled or welded shapes, plate girders and cover-plated beams are proportioned on the basis of flexural strength of the gross section:

(a) If $$0.75F_u A_{fn} \geq 0.9Fy A_{fg} \qquad \textbf{(2.10-1)}$$

no deduction shall be made for bolt or rivet holes in either flange, where

A_{fg} = gross flange area, mm^2

A_{fn} = net tension flange area calculated in accordance with the provisions of Section 2.1 and 2.2, mm^2

F_u = specified minimum tensile strength, MPa

(b) If $$0.75F_u A_{fn} < 0.9F_y A_{fg} \qquad \textbf{(2.10-2)}$$

the member flexural properties shall be based on an effective tension flange area A_{fe}

$$A_{fe} = \frac{5}{6}\frac{F_u}{F_y}A_{fn} \qquad \textbf{(2.10-3)}$$

and the maximum flexural strength shall be based on the elastic section modulus.

Other design requirements for *proper* proportioning of beams and girders are as follows:

Hybrid girders shall be proportioned by the flexural strength of their gross section, subject to the applicable provisions in Section 7.1, provided they are not required to resist an axial force greater than ϕ_b times $0.15F_{yf} A_g$, where F_{yf} is the specified minimum yield stress of the flange material and A_g is the gross area. No limit is placed on the web stresses produced by the applied bending moment for which a hybrid girder is designed, except as provided in Section 11.3. To qualify as hybrid girders, the flanges at any given section shall have the same cross-sectional area and be made of the same grade of steel.

Flanges of welded beams or girders may be varied in thickness or width by splicing a series of plates or by the use of cover plates.

The total cross-sectional area of cover plates of bolted or riveted girders shall not exceed 70 percent of the total flange area.

High-strength bolts, rivets, or welds connecting flange to web, or cover plate to flange, shall be *proportioned to resist the total horizontal shear* resulting from the bending forces on the girder. The longitudinal distribution of these bolts, rivets, or intermittent welds shall be in proportion to the intensity of the shear. However, the longitudinal spacing shall not exceed the maximum permitted for compression or tension members in Section 5.4 or 4.2, respectively. Bolts, rivets, or welds connecting flange to web shall also be proportioned to transmit to the web any loads applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

Partial length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by high-strength bolts in a slip-critical connection, rivets, or fillet welds. The attachment shall be adequate, at the applicable design strength given in Sections 10.2.2, 10.3.8 or 11.3 to develop the cover plate's portion of the flexural design strength in the beam or girder at the theoretical cutoff point.

For *welded cover plates*, the welds connecting the cover plate termination to the beam or girder shall have continuous welds along both edges of the cover plate in the length a', defined below, and shall be adequate, at the applicable design strength, to develop the cover plate's portion of the design strength in the beam or girder at the distance a' from the end of the cover plate.

(a) When there is a continuous weld equal to or larger than three-fourths of the plate thickness across the end of the plate

$$a' = w \tag{2.10-4}$$

where:

w = width of cover plate, mm

(b) When there is a continuous weld smaller than three-fourths of the plate thickness across the end of the plate

$$a' = 1.5w \tag{2.10-5}$$

(c) When there is no weld across the end of the plate

$$a' = 2w \tag{2.10-6}$$

CHAPTER 3
FRAMES AND OTHER STRUCTURES

This chapter contains general requirements for stability of the structure as a whole.

SECTION 3.1
SECOND ORDER EFFECTS

Second order ($P\Delta$) effects shall be considered in the design of frames.

3.1.1 **Design by Plastic Analysis.** In structures designed on the basis of plastic analysis, as limited in Section 1.5.1, the required flexural strength M_u shall be determined from a second-order plastic analysis that satisfies the requirements of Section 3.2.

3.1.2 **Design by Elastic Analysis.** In structures designed on the basis of elastic analysis, M_u for beam-columns, connections, and connected members shall be determined from a second-order elastic analysis or from the following approximate second-order analysis procedure:

$$M_u = B_1 M_{nt} + B_2 M_{lt} \qquad \textbf{(3.1-1)}$$

where

M_{nt} = required flexural strength in member assuming there is no lateral translation of the frame, N.mm

M_{lt} = required flexural strength in member as a result of lateral translation of the frame only, N.mm

$$B_1 = \frac{C_m}{(1 - P_u / P_{e1})} \geq 1 \qquad \textbf{(3.1-2)}$$

$$P_{e1} = \frac{\pi^2 EI}{(KL)^2}, \text{ N}$$

where I is the moment of inertia in the plane of bending and K is the effective length factor in the plane of bending determined in accordance with Section 3.2.1, for the braced frame.

P_u = required axial compressive strength for the member under consideration, N

C_m = a coefficient based on elastic first-order analysis assuming no lateral translation of the frame whose value shall be taken as follows:

(a) For compression members not subject to transverse loading between their supports in the plane of bending,

$$C_m = 0.6 - 0.4(M_1 / M_2) \qquad \textbf{(3.1-3)}$$

where M_1 / M_2 is the ratio of the smaller to larger moments at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1 / M_2 is positive when the member is bent in reverse curvature, negative when bent in single curvature.

(b) For compression members subjected to transverse loading between their supports, the value of C_m shall be determined either by rational analysis (see commentary) or by the use of the following values:

For members whose ends are restrained. . . . . . . $C_m = 0.85$

For members whose ends are unrestrained. . . . . . $C_m = 1.00$

$$B_2 = \frac{1}{1-\Sigma P_u\left(\dfrac{\Delta_{oh}}{\Sigma HL}\right)} \tag{3.1-4}$$

or

$$B_2 = \frac{1}{1-\left(\dfrac{\Sigma P_u}{\Sigma P_{e2}}\right)} \tag{3.1-5}$$

$\sum P_u$ = required axial strength of all columns in a story, N

Δ_{oh} = lateral inter-story deflection, mm

$\sum H$ = sum of all story horizontal forces producing Δ_{oh}, N

L = story height, mm

P_{e_2} = $\dfrac{\pi^2 EI}{(KL)^2}$, N

where I is the moment of inertia in the plane of bending and K is the effective length factor in the plane of bending determined in accordance with Section 3.2.2, for the unbraced frame.

SECTION 3.2
FRAME STABILITY

3.2.1 **Braced Frames.** In trusses and frames where lateral stability is provided by diagonal bracing, shear walls, or equivalent means, the effective length factor K for compression members shall be taken as unity, unless structural analysis shows that a smaller value may be used.

The vertical bracing system for a braced multistory frame shall be determined by structural analysis to be adequate to prevent buckling of the structure and to maintain the lateral stability of the structure, including the overturning effects of drift under the factored load combinations stipulated in Section 1.4.

The vertical bracing system for a braced multistory frame may be considered to function together with in-plane shear-resisting exterior and interior walls, floor slabs, and roof decks, which are properly secured to the structural frames. The columns, girders, beams, and diagonal members, when used as the vertical bracing system, may be considered to comprise a vertically cantilevered simply connected truss in the analyses for frame buckling and lateral stability. Axial deformation of all members in the vertical bracing system shall be included in the lateral stability analysis.

3.2.1.1 **Design by Plastic Analysis.** In braced frames designed on the basis of plastic analysis, as limited in Section 1.5.1, the axial force in these members caused by factored gravity plus factored horizontal loads shall not exceed 0.85 ϕ_c times $A_g F_y$.

3.2.2 **Unbraced Frames.** In frames where lateral stability depends upon the bending stiffness of rigidly connected beams and columns, the effective length factor K of compression members shall be determined by structural analysis. The destabilizing effects of gravity loaded columns whose simple connections to the frame do not provide resistance to lateral loads shall be included in the design of

the moment-frame columns. Stiffness reduction adjustment due to column inelasticity is permitted.

Analysis of the required strength of unbraced multistory frames shall include the effects of frame instability and column axial deformation under the factored load combinations stipulated in Section 1.4.

3.2.2.1 **Design by Plastic Analysis.** In unbraced frames designed on the basis of plastic analysis, as limited in Section 1.5.1, the axial force in the columns caused by factored gravity plus factored horizontal loads shall not exceed 0.75 ϕ_c times $A_g F_y$.

SECTION 3.3
STABILITY BRACING

3.3.1 **Scope.** These requirements address the minimum brace strength and stiffness necessary to ensure member design strengths based on the unbraced length between braces with an effective length factor K equal to unity. Bracing is assumed to be perpendicular to the member(s) to be braced; for inclined or diagonal bracing, the brace strength (force or moment) and stiffness (force per unit displacement or moment per unit rotation) must be adjusted for the angle of inclination. The evaluation of the stiffness furnished by a brace shall include its member and geometric properties, as well as the effects of connections and anchoring details.

Two general types of bracing systems are considered, relative and nodal. A relative brace controls the movement of the brace point with respect to adjacent braced points. A nodal brace controls the movement at the braced point without direct interaction with adjacent braced points. The strength and stiffness furnished by the stability bracing shall not be less than the required limits. A second order analysis that includes an initial out-of-plumbness of the structure or out-of-straightness of the member to obtain brace strength and stiffness can be used in lieu of the requirements of this section.

3.3.2 **Frames.** In braced frames where lateral stability is provided by diagonal bracing, shear walls, or other equivalent means, the required story or panel bracing shear force is:

$$P_{br} = 0.004\Sigma P_u \quad \textbf{(3.3-1)}$$

The required story or panel shear stiffness is:

$$\beta_{br} = \frac{2\Sigma P_u}{\phi L} \quad \textbf{(3.3-2)}$$

where

ϕ = 0.75

ΣP_u = summation of the factored column axial loads in the story or panel supported by the bracing, N

L = story height or panel spacing, mm

These story stability requirements shall be combined with the lateral forces and drift requirements from other sources, such as wind or seismic loading.

3.3.3 **Columns.** An individual column can be braced at intermediate points along its length by relative or nodal bracing systems. It is assumed that nodal braces are equally spaced along the column.

(a) *Relative Bracing*

The required brace strength is:

$$P_{br} = 0.004P_u \tag{3.3-3}$$

The required brace stiffness is:

$$\beta_{br} = \frac{2P_u}{\phi L_b} \tag{3.3-4}$$

where

ϕ = 0.75

P_u = required compressive strength, N

L_b = distance between braces, mm

(b) *Nodal Bracing*

The required brace strength is:

$$P_{br} = 0.01P_u \tag{3.3-5}$$

The required brace stiffness is:

$$\beta_{br} = \frac{8P_u}{\phi L_b} \tag{3.3-6}$$

where

ϕ = 0.75

When the actual spacing of braced points is less than L_q, where L_q is the maximum unbraced length for the required column force with K equal to one, then L_b in Equations 3.3-4 and 3.3-6 is permitted to be taken equal to L_q.

3.3.4 **Beams.** Beam bracing must prevent the relative displacement of the top and bottom flanges, i.e. twist of the section. Lateral stability of beams shall be provided by lateral bracing, torsional bracing, or a combination of the two. In members subjected to double curvature bending, the inflection point shall not be considered a brace point.

3.3.4.1 **Lateral Bracing.** Bracing shall be attached near the compression flange, except for a cantilevered member, where an end brace shall be attached near the top (tension) flange. Lateral bracing shall be attached to both flanges at the brace point near the inflection point for beams subjected to double curvature bending along the length to be braced.

(a) *Relative Bracing*

The required brace strength is:

$$P_{br} = 0.008M_u C_d / h_o \tag{3.3-7}$$

The required brace stiffness is:

$$\beta_{br} = \frac{4M_u C_d}{\phi L_b h_o} \tag{3.3-8}$$

where

ϕ = 0.75

M_u = required flexural strength, N.mm

h_o = distance between flange centroids, mm

C_d = 1.0 for bending in single curvature; 2.0 for double curvature;

C_d = 2.0 only applies to the brace closest to the inflection point.

L_b = distance between braces, mm

(b) *Nodal Bracing*

The required brace strength is:

$$P_{br} = 0.02 M_u C_d / h_o \qquad \textbf{(3.3-9)}$$

The required brace stiffness is:

$$\beta_{br} = \frac{10 M_u C_d}{\phi L_b h_o} \qquad \textbf{(3.3-10)}$$

where

ϕ = 0.75

When the actual spacing of braced points is less than L_q, the maximum unbraced length for M_u, then L_b in Equations 3.3-8 and 3.3-10 shall be permitted to be taken equal to L_q.

3.3.4.2 **Torsional Bracing.** Torsional bracing can be nodal or continuous along the beam length. The bracing can be attached at any cross-sectional location and need not be attached near the compression flange. The connection between a torsional brace and the beam must be able to support the required moment given below.

(a) *Nodal Bracing*

The required bracing moment is:

$$M_{br} = \frac{0.024 M_u L}{n C_b L_b} \qquad \textbf{(3.3-11)}$$

The required cross-frame or diaphragm bracing stiffness is:

$$\beta_{Tb} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{sec}}\right)} \qquad \textbf{(3.3-12)}$$

where

$$\beta_T = \frac{2.4 L M_u^2}{\phi n E I_y C_b^2} \qquad \textbf{(3.3-13)}$$

$$\beta_{sec} = \frac{3.3E}{h_o}\left(\frac{1.5 h_o t_w^3}{12} + \frac{t_s b_s^3}{12}\right) \qquad \textbf{(3.3-14)}$$

ϕ = 0.75

L = span length, mm

n = number of nodal braced points within the span

E = 200,000 MPa

I_y = out-of-plane moment of inertia, mm^4

C_b = is a modification factor defined in Chapter 6

t_w = beam web thickness, mm

t_s = web stiffener thickness, mm

b_s = stiffener width for one-sided stiffeners (use twice the individual stiffener width for pairs of stiffeners), mm

β_T = brace stiffness excluding web distortion, N-mm/radian

β_{sec} = web distortional stiffness, including the effect of web transverse stiffeners, if any, N-mm/radian

If $\beta_{sec} < \beta_T$, Equation 3.3-12 is negative, which indicates that torsional beam bracing will not be effective due to inadequate web distortional stiffness.

When required, the web stiffener shall extend the full depth of the braced member and shall be attached to the flange if the torsional brace is also attached to the flange. Alternatively, it shall be permissible to stop the stiffener short by a distance equal to $4t_w$ from any beam flange that is not directly attached to the torsional brace. When the actual spacing of braced points is less than L_q, then L_b in Equation 3.3-11 shall be permitted to be taken equal to L_q.

(b) *Continuous Torsional Bracing*

For continuous bracing, use Equations 3.3-11, 3.3-12 and 3.3-13 with L/n taken as 1.0; the bracing moment and stiffness are given per unit span length. The distortional stiffness for an unstiffened web is

$$\beta_{sec} = \frac{3.3Et_w^3}{12h_o} \qquad \textbf{(3.3-15)}$$

CHAPTER 4
TENSION MEMBERS

This chapter applies to prismatic members subject to axial tension caused by static forces acting through the centroidal axis. For members subject to combined axial tension and flexure, see Section 8.1.1. For threaded rods, see Section 10.3. For block shear rupture strength at end connections of tension members, see Section 10.4.3. For the design tensile strength of connecting elements, see Section 10.5.2. For members subject to fatigue, see Section 11.3.

SECTION 4.1
DESIGN TENSILE STRENGTH

The design strength of tension members $\phi_t P_n$, shall be the lower value obtained according to the limit states of yielding in the gross section and fracture in the net section.

(a) For yielding in the gross section:

$$\phi_t = 0.90$$

$$P_n = F_y A_g \tag{4.1-1}$$

(b) For fracture in the net section:

$$\phi_t = 0.75$$

$$P_n = F_u A_e \tag{4.1-2}$$

where

A_e = effective net area, mm^2

A_g = gross area of member, mm^2

F_y = specified minimum yield stress, MPa

F_u = specified minimum tensile strength, MPa

When members without holes are fully connected by welds, the effective net section used in Equation 4.1-2 shall be defined as Section 2.3. When holes are present in a member with welded-end connections, or at the welded connection in the case of plug or slot welds, the net section through the holes shall be used in Equation 4.1-2.

SECTION 4.2
BUILT-UP MEMBERS

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, see Section 10.3.5.

The longitudinal spacing of connectors between components should preferably limit the slenderness ratio in any component between the connectors to 300.

Either perforated cover plates or tie plates without lacing are permitted to be used on the open sides of built-up tension members. Tie plates shall have a length not less than two-thirds the distance between the lines of welds or fasteners connecting them to the components of the member. The thickness of such tie plates shall not be less than one-fiftieth of the distance between these lines. The longitudinal spacing of intermittent welds or fasteners at tie plates shall not exceed 150 mm. The spacing of tie plates shall be such that the slenderness ratio of any component in the length between tie plates should preferably not exceed 300.

SECTION 4.3
PIN-CONNECTED MEMBERS AND EYEBARS

4.3.1 Pin-Connected Members

4.3.1.1 Design Strength

The design strength of a pin-connected member, ϕP_n shall be the lowest value of the following limit states:

(a) Tension on the net effective area:

$$\phi = 0.75$$

$$P_n = 2tb_{eff}F_u \quad \textbf{(4.3-1)}$$

(b) Shear on the effective area:

$$\phi = 0.75$$

$$P_n = 0.6A_{sf}F_u \quad \textbf{(4.3-2)}$$

(c) For bearing on the projected area of the pin, see Section 10.8.

(d) For yielding in the gross section, use Equation 4.1-1.

where

A_{sf} = $2t(a + d/2)$, mm^2

a = shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the force, mm

b_{eff} = $2t + 16$, mm but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force

d = pin diameter, mm

t = thickness of plate, mm

4.3.1.2 Detailing Requirements. The pin hole shall be located midway between the edges of the member in the direction normal to the applied force. When the pin is expected to provide for relative movement between connected parts while under full load, the diameter of the pin hole shall not be more than 1 mm greater than the diameter of the pin.

The width of the plate beyond the pin hole shall not be less than $2b_{eff} + d$ and the minimum extension, a, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than 1.33 x b_{eff}.

The corners beyond the pinhole are permitted to be cut at 45^o to the axis of the member, provided the net area beyond the pinhole, on a plane perpendicular to the cut, is not less than that required beyond the pinhole parallel to the axis of the member.

4.3.2 Eyebars

4.3.2.1 Design Strength. The design strength of eyebars shall be determined in accordance with 4.1, with A_g taken as the cross-sectional area of the body.
For calculation purposes, the width of the body of the eyebars shall not exceed eight times its thickness.

4.3.2.2 **Detailing Requirements.** Eyebars shall be of uniform thickness, without reinforcement at the pin holes, and have circular heads whose periphery is concentric with the pin hole.

The radius of transition between the circular head and the eyebar body shall not be less than the head diameter.
The pin diameter shall not be less than seven-eighths times the eyebar body width, and the pin hole diameter shall not be more than 1 mm greater than the pin diameter.

For steels having F_y greater than 485 MPa, the hole diameter shall not exceed five times the plate thickness and the width of the eyebar body shall be reduced accordingly.

A thickness of less than 12 mm is permissible only if external nuts are provided to tighten pin plates and filler plates into snug contact. The width from the hole edge to the plate edge perpendicular to the direction of applied load shall be greater than two-thirds and, for the purpose of calculation, not more than three-fourths times the eyebar body width.

CHAPTER 5
COLUMN AND OTHER COMPRESSION MEMBERS

This chapter applies to compact and non-compact prismatic members subject to axial compression through the centroidal axis. For members subject to combined axial compression and flexure, see Section 8.1.2. For members with slender compression elements, see Section 2.5.3. For tapered members, see Section 6.3.

SECTION 5.1
EFFECTIVE LENGTH AND SLENDERNESS LIMITATIONS

5.1.1 **Effective Length.** The effective length factor K shall be determined in accordance with Section 3.2.

5.1.2 **Design by Plastic Analysis.** Design by plastic analysis, as limited in Section 1.5.1, is permitted if the column slenderness parameter λ_c does not exceed $1.5K$.

SECTION 5.2
DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING

5.2.1 **Width-Thickness Ratio of Elements Less than or Equal to λ_r.** The design strength for flexural buckling of compression members whose elements have width-thickness ratios less than or equal to λ_r from Section 2.5.1 is $\phi_c P_n$:

$$\phi_c = 0.85$$

$$P_n = A_g F_{cr} \tag{5.2-1}$$

(a) For $\lambda_c \le 1.5$

$$F_{cr} = \left(0.658^{\lambda_c^2}\right) F_y \tag{5.2-2}$$

(b) For $\lambda_c > 1.5$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y \tag{5.2-3}$$

where

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}} \tag{5.2-4}$$

A_g = gross area of member, mm^2
F_y = specified minimum yield stress, MPa
E = modulus of elasticity, MPa
K = effective length factor
l = laterally unbraced length of member, mm
r = governing radius of gyration about the axis of buckling, mm

5.2.2 **Width-Thickness Ratio of Elements Exceeds λ_r**

For members whose elements do not meet the requirements of Section 2.5.1, see 2.5.3.

SECTION 5.3
DESIGN COMPRESSIVE STRENGTH FOR FLEXURAL-TORSIONAL BUCKLING

5.3.1 **Width-Thickness Ratios of Elements Less than or Equal to λ_r.** The design strength for flexural-torsional buckling of double-angle and tee-shaped compression members whose elements have width-thickness ratios less than λ_r from Section 2.5.1 is $\phi_c P_n$:

where

$$\phi_c = 0.85$$

$$P_n = A_g F_{crft} \quad \textbf{(5.3-1)}$$

$$F_{crft} = \left(\frac{F_{cry} + F_{crz}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{(F_{cry} + F_{crz})^2}}\right] \quad \textbf{(5.3-2)}$$

$F_{crz} = \dfrac{GJ}{A\bar{r}_o^2}$

$\bar{r}_o$ = polar radius of gyration about shear center, mm (see Equation 5.3-10)

$H = 1 - \dfrac{y_o^2}{\bar{r}_o^2}$

y_o = distance between shear center and centroid, mm

F_{cry} = is determined according to Section 5.2 for flexural buckling about the y-axis of symmetry for $\lambda_c = \dfrac{Kl}{r_y \pi}\sqrt{\dfrac{F_y}{E}}$.

For double-angle and tee-shaped members whose elements do not meet the requirements of Section 2.5.1, see 2.5.3 to determine F_{cry} for use in Equation 5.3-2.

5.3.2 **Width-Thickness Ratio of Elements Exceeds λ_r.** This section applies to the strength of doubly symmetric columns with thin plate elements, and singly symmetric and unsymmetric columns for the limit states of flexural-torsional and torsional buckling.

The design strength of compression members determined by the limit states of torsional and flexural-torsional buckling is $\phi_c P_n$,

where

ϕ_c = 0.85

P_n = nominal resistance in compression, N

= $A_g F_{cr}$ **(5.3-3)**

A_g = gross area of cross section, mm^2

The nominal critical stress F_{cr} is determined as follows:

(a) For $\lambda_e \sqrt{Q} \le 1.5$:

$$F_{cr} = Q(0.658^{Q\lambda_e^2})F_y \quad \textbf{(5.3-4)}$$

(b) For $\lambda_e \sqrt{Q} > 1.5$:

$$F_{cr} = \left[\frac{0.877}{\lambda_e^2}\right] F_y \qquad \textbf{(5.3-5)}$$

where

$$\lambda_e = \sqrt{F_y / F_e} \qquad \textbf{(5.3-6)}$$

Q = 1.0 for elements meeting the width-thickness ratios λ_r of Section 2.5.1
= $Q_s Q_a$ for elements not meeting the width-thickness ratios λ_r of Section 2.5.1 and determined in accordance with the provisions of Section 2.5.3

The critical torsional or flexural-torsional elastic buckling stress F_e is determined as follows:

(a) For doubly symmetric shapes:

$$F_e = \left[\frac{\pi^2 E C_w}{(K_z l)^2} + GJ\right] \frac{1}{I_x + I_y} \qquad \textbf{(5.3-7)}$$

(b) For singly symmetric shapes where y is the axis of symmetry:

$$F_e = \frac{F_{ey} + F_{ez}}{2H}\left(1 - \sqrt{1 - \frac{4 F_{ey} F_{ez} H}{(F_{ey} + F_{ez})^2}}\right) \qquad \textbf{(5.3-8)}$$

(c) For unsymmetric shapes, the critical flexural-torsional elastic buckling stress F_e is the lowest root of the cubic equation

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2 (F_e - F_{ey})\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2 (F_e - F_{ex})\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0 \qquad \textbf{(5.3-9)}$$

where

K_z = effective length factor for torsional buckling
G = shear modulus, MPa
C_w = warping constant, mm^6
J = torsional constant, mm^4
I_x, I_y = moment of inertia about the principal axes, mm^4
x_o, y_o = coordinates of shear center with respect to the centroid, mm

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A} \qquad \textbf{(5.3-10)}$$

$$H = 1 - \left(\frac{x_o^2 + y_o^2}{\bar{r}_o^2}\right) \qquad \textbf{(5.3-11)}$$

$$F_{ex} = \frac{\pi^2 E}{(K_x l / r_x)^2} \qquad \textbf{(5.3-12)}$$

$$F_{ey} = \frac{\pi^2 E}{(K_y l / r_y)^2} \qquad \textbf{(5.3-13)}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z l)^2} + GJ\right) \frac{1}{A \bar{r}_o^2} \qquad \textbf{(5.3-14)}$$

A = cross-sectional area of member, mm^2
l = unbraced length, mm

K_x , K_y = effective length factors in x and y directions

r_x , r_y = radii of gyration about the principal axes, mm

$\bar{r}_o$ = polar radius of gyration about the shear center, mm

SECTION 5.4
BUILT-UP MEMBERS

5.4.1 **Design Strength.** The design strength of built-up members composed of two or more shapes shall be determined in accordance with Section 5.2 and Section 5.3 subject to the following modification. If the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, Kl/r is replaced by $(Kl/r)_m$ determined as follows:

(a) For intermediate connectors that are snug-tight bolted:

$$\left(\frac{Kl}{r}\right)_m = \sqrt{\left(\frac{Kl}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad \textbf{(5.4-1)}$$

(b) For intermediate connectors that are welded or fully tensioned bolted:

$$\left(\frac{Kl}{r}\right)_m = \sqrt{\left(\frac{Kl}{r}\right)_o^2 + 0.82\frac{\alpha^2}{(1+\alpha^2)}\left(\frac{a}{r_{ib}}\right)^2} \qquad \textbf{(5.4-2)}$$

where

$\left(\frac{Kl}{r}\right)_o$ = column slenderness of built-up member acting as a unit

$\left(\frac{Kl}{r}\right)_m$ = modified column slenderness of built-up member

a = distance between connectors, mm

r_i = minimum radius of gyration of individual component, mm

r_{ib} = radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, mm

α = separation ratio = $h/2r_{ib}$

h = distance between centroids of individual components perpendicular to the member axis of buckling, mm

5.4.2 **Detailing Requirements.** At the ends of built-up compression members bearing on base plates or milled surfaces, all components in contact with one another shall be connected by a weld having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to 1½ times the maximum width of the member.

Along the length of built-up compression members between the end connections required above, longitudinal spacing for intermittent welds, bolts, or rivets shall be adequate to provide for the transfer of the required forces. For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, see Section 10.3.5. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times $0.75\sqrt{E/F_y}$, nor 305 mm, when intermittent welds are provided along the edges of the components or when fasteners are provided on all gage lines at each section.

When fasteners are staggered, the maximum spacing on each gage line shall not exceed the thickness of the thinner outside plate times $1.12\sqrt{E/F_y}$ nor 460 mm.

Individual components of compression members composed of two or more shapes shall be connected to one another at intervals, a, such that the effective slenderness ratio Ka/r_i of each of the component shapes, between the connectors, does not exceed three-fourths times the governing slenderness ratio of the built-up member. The least radius of gyration r_i shall be used in computing the slenderness ratio of each component part. The end connection shall be welded or fully tensioned bolted with clean mill scale or blast-cleaned faying surfaces with Class A coatings.

Open sides of compression members built up from plates or shapes shall be provided with continuous cover plates perforated with a succession of access holes. The unsupported width of such plates at access holes, as defined in Section 2.5.1, is assumed to contribute to the design strength provided that:

(1) The width-thickness ratio conforms to the limitations of Section 2.5.1.

(2) The ratio of length (in direction of stress) to width of hole shall not exceed two.

(3) The clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds.

(4) The periphery of the holes at all points shall have a minimum radius of 38 mm.

As an alternative to perforated cover plates, lacing with tie plates is permitted at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In main members providing design strength, the end tie plates shall have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate shall aggregate not less than one-third the length of the plate. In bolted and riveted construction, the spacing in the direction of stress in tie plates shall be not more than six diameters and the tie plates shall be connected to each segment by at least three fasteners.

Lacing, including flat bars, angles, channels, or other shapes employed as lacing, shall be so spaced that l/r of the flange included between their connections shall not exceed the governing slenderness ratio for the member as a whole. Lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to two percent of the compressive design strength of the member. The l/r ratio for lacing bars arranged in single systems shall not exceed 140. For double lacing this ratio shall not exceed 200. Double lacing bars shall be joined at the intersections. For lacing bars in compression, l is permitted to be taken as the unsupported length of the lacing bar between welds or fasteners connecting it to the components of the built-up member for single lacing, and 70 percent of that distance for double lacing. The inclination of lacing bars to the axis of the member shall preferably be not less than 60° for single lacing and 45° for double lacing. When the distance between the lines of welds or fasteners in the flanges is more than 380 mm, the lacing shall preferably be double or be made of angles.

For additional spacing requirements, see Section 10.3.

SECTION 5.5
CONNECTIONS FOR PIN-CONNECTED COMPRESSION MEMBERS

Pin connections of pin-connected compression members shall conform to the requirements of Sections 4.3.1 and 4.3.2, except Equations 4.3-1 and 4.3-2 do not apply.

CHAPTER 6
BEAMS AND OTHER FLEXURAL MEMBERS

This chapter applies to compact and noncompact prismatic members subject to flexure and shear. For member subject to combined flexure and axial force, see Section 8.1. For members subject to fatigue, see Section 11.3. For members with slender compression elements, see Section 2.5. For web-tapered members, see Section 6.3. For members with slender web elements (plate girders), see Chapter 7.

SECTION 6.1
DESIGN FOR FLEXURE

The nominal flexural strength M_n is the lowest value obtained according to the limit states of: (a) yielding; (b) lateral-torsional buckling; (c) flange local buckling; and (d) web local buckling. For laterally braced compact beams with $L_b \leq L_p$, only the limit state of yielding is applicable. For unbraced compact beams and noncompact tees and double angles, only the limit states of yielding and lateral-torsional buckling are applicable. The lateral-torsional buckling limit state is not applicable to members subject to bending about the minor axis, or to square or circular shapes.

This section applies to homogeneous and hybrid shapes with at least one axis of symmetry and which are subject to simple bending about one principal axis. For simple bending, the beam is loaded in a plane parallel to a principal axis that passes through the shear center or the beam is restrained against twisting at load points and supports. Only the limit states of yielding and lateral-torsional buckling are considered in this section. The lateral-torsional buckling provisions are limited to doubly symmetric shapes, channels, double angles, and tees. For lateral-torsional buckling of other singly symmetric shapes and for the limit states of flange local buckling and web local buckling of noncompact or slender-element sections, see Section 6.1.2.4 of this Chapter.

6.1.1 **Yielding.** The flexural design strength of beams, determined by the limit state of yielding, is $\phi_b M_n$:

$$\phi_b = 0.90$$

$$M_n = M_p \qquad \textbf{(6.1-1)}$$

where

M_p = plastic moment ($= F_y Z \leq 1.5\, M_y$ for homogeneous sections), N-mm

M_y = moment corresponding to onset of yielding at the extreme fiber from an elastic stress distribution ($= F_y S$ for homogeneous section and $F_{yf} S$ for hybrid sections), N-mm

See Section 2.10 for further limitations on M_n where there are holes in the tension flange.

6.1.2 **Lateral-Torsional Buckling.** This limit state is only applicable to members subject to major axis bending. The flexural design strength, determined by the limit state of lateral-torsional buckling, is $\phi_b M_n$:

ϕ_b = 0.90

M_n = nominal flexural strength determined as follows

6.1.2.1 **Doubly Symmetric Shapes and Channels with $L_b \leq L_r$.** The nominal flexural strength is:

$$M_n = C_b\left[M_p - \left(M_p - M_r\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \qquad \textbf{(6.1-2)}$$

where

L_b = distance between points braced against lateral displacement of the compression flange, or between points braced to prevent twist of the cross section, mm

L_p = limiting laterally unbraced length as defined below, mm

L_r = limiting laterally unbraced length as defined below, mm

M_r = limiting buckling moment as defined below, N-mm

In the above equation, C_b is a modification factor for non-uniform moment diagrams where, when both ends of the beam segment are braced:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \qquad \textbf{(6.1-3)}$$

where

$M_{\max}$ = absolute value of maximum moment in the unbraced segment, N-mm

M_A = absolute value of moment at quarter point of the unbraced segment, N-mm

M_B = absolute value of moment at centerline of the unbraced beam segment, N-mm

M_C = absolute value of moment at three-quarter point of the unbraced beam segment, N-mm

C_b = is permitted to be conservatively taken as 1.0 for all cases. Equations 6.1-4 and 6.1-6 are conservatively based on C_b = 1.0. For cantilevers or overhangs where the free end is unbraced, C_b = 1.0.

The limiting unbraced length, L_p, shall be determined as follows.

(a) For I-shaped members including hybrid sections and channels:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_{yf}}} \qquad \textbf{(6.1-4)}$$

(b) For solid rectangular bars and box sections:

$$L_p = \frac{0.13 r_y E}{M_p}\sqrt{JA} \qquad \textbf{(6.1-5)}$$

where

A = cross-sectional area, mm^2

J = torsional constant, mm^4

The limiting laterally unbraced length L_r and the corresponding buckling moment M_r shall be determined as follows.

(a) For doubly symmetric I-shaped members and channels:

$$L_r = \frac{r_y X_1}{F_L}\sqrt{1+\sqrt{1+X_2 F_L^2}} \tag{6.1-6}$$

$$M_r = F_L S_x \tag{6.1-7}$$

where

$$X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}} \tag{6.1-8}$$

$$X_2 = 4\frac{C_w}{I_y}\left(\frac{S_x}{GJ}\right)^2 \tag{6.1-9}$$

S_x = section modulus about major axis, mm^3
E = modulus of elasticity of steel, 200,000 MPa
G = shear modulus of elasticity of steel, 77,200 MPa
F_L = smaller of (F_{yf} - F_r) or F_{yw}, MPa
F_r = compressive residual stress in flange; 69 MPa for rolled shapes, 114 MPa for welded built-up shapes
F_{yf} = yield stress of flange, MPa
F_{yw} = yield stress of web, MPa
I_y = moment of inertia about y-axis, mm^4
C_w = warping constant, mm^6

(b) For solid rectangular bars and box sections:

$$L_r = \frac{2r_y E\sqrt{JA}}{M_r} \tag{6.1-10}$$

$$M_r = F_{yf} S_x \tag{6.1-11}$$

6.1.2.2 Doubly Symmetric Shapes and Channels with $L_b > L_r$

The nominal flexural strength is:

$$M_n = M_{cr} \le M_p \tag{6.1-12}$$

where

(a) For doubly symmetric I-shaped members and channels:

$$\begin{aligned} M_{cr} &= C_b \frac{\pi}{L_b}\sqrt{EI_y GJ + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w} \\ &= \frac{C_b S_x X_1\sqrt{2}}{L_b / r_y} 1 + \frac{X_1^2 X_2}{2(L_b / r_y)^2} \end{aligned} \tag{6.1-13}$$

(b) For solid rectangular bars and symmetric box sections:

$$M_{cr} = \frac{57000 C_b \sqrt{JA}}{L_b / r_y} \quad \textbf{(6.1-14)}$$

6.1.2.3 **Tees and Double Angles**. For tees and double-angle beams loaded in the plane of symmetry:

$$M_n = M_{cr} = \frac{\pi \sqrt{EI_y GJ}}{L_b} \left[B + \sqrt{1 + B^2} \right] \quad \textbf{(6.1-15)}$$

where

$M_n \leq 1.5 M_y$ for stems in tension

$M_n \leq 1.0 M_y$ for stems in compression

$$B = \pm 2.3 (d / L_b) \sqrt{I_y / J} \quad \textbf{(6.1-16)}$$

The plus sign for B applies when the stem is in tension and the minus sign applies when the stem is in compression. If the tip of the stem is in compression anywhere along the unbraced length, use the negative value of B.

6.1.2.4 Yielding, lateral-torsional buckling, flange local buckling, and web local buckling of singly symmetric shapes other than those addressed in Sections 6.1.1 and 6.1.2 of this Chapter and of noncompact or slender-element sections:

The nominal flexural strength M_n is the lowest value obtained according to the following limit states determined as follows:

(a) For $\lambda \leq \lambda_p$:

$$M_n = M_p \quad \textbf{(6.1-17)}$$

(b) For $\lambda_p < \lambda \leq \lambda_r$:

For the limit state of lateral-torsional buckling:

$$M_n = C_b \left[M_p - (M_p - M_r) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \right] \leq M_p \quad \textbf{(6.1-18)}$$

For the limit states of flange and web local buckling:

$$M_n = M_p - (M_p - M_r) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \quad \textbf{(6.1-19)}$$

(c) For $\lambda > \lambda_r$:

For the limit states of lateral-torsional buckling and flange local buckling:

$$M_n = M_{cr} = S F_{cr} \leq M_p \quad \textbf{(6.1-20)}$$

The terms used in the above equations are:

M_n = nominal flexural strength, N-mm

M_p = $F_y Z$, plastic moment $\leq 1.5\, F_y S$, N-mm

M_{cr} = buckling moment, N-mm

M_r = limiting buckling moment (equal to M_{cr} when $\lambda = \lambda_r$), N-mm

λ = controlling slenderness parameter

= minor axis slenderness ratio L_b / r_y for lateral-torsional buckling

= flange width-thickness ratio b / t for flange local buckling as defined in Section 2.5.1

= web depth-thickness ratio h / t_w for web local buckling as defined in Section 2.5.1

λ_p = largest value of λ for which $M_n = M_p$

λ_r = largest value of λ for which buckling is inelastic

F_{cr} = critical stress, MPa

C_b = bending coefficient dependent on moment gradient, see Section 6.1.2.1, Equation 6.1-3

S = section modulus, mm^3

L_b = laterally unbraced length, mm

r_y = radius of gyration about minor axis, mm

6.1.3 **Design by Plastic Analysis.** Design by plastic analysis, as limited in Section 1.5.1, is permitted for a compact section member bent about the major axis when the laterally unbraced length L_b of the compression flange adjacent to plastic hinge locations associated with the failure mechanism does not exceed L_{pd}, determined as follows.

(a) For doubly symmetric and singly symmetric I-shaped members with the compression flange equal to or larger than the tension flange (including hybrid members) loaded in the plane of the web:

$$L_{pd} = \left[0.12 + 0.076\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right) r_y \qquad \textbf{(6.1-21)}$$

where

F_y = specified minimum yield stress of the compression flange, MPa

M_1 = smaller moment at end of unbraced length of beam, N-mm

M_2 = larger moment at end of unbraced length of beam, N-mm

r_y = radius of gyration about minor axis, mm

(M_1 / M_2) = is positive when moments cause reverse curvature and negative for single curvature

(b) For solid rectangular bars and symmetric box beams:

$$L_{pd} = \left[0.17 + 0.10\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right) r_y \geq 0.10\left(\frac{E}{F_y}\right) r_y \qquad \textbf{(6.1-22)}$$

There is no limit on L_b for members with circular or square cross sections or for any beam bent about its minor axis.

In the region of the last hinge to form, and in regions not adjacent to a plastic hinge, the flexural design strength shall be determined in accordance with Section 6.1.2.

SECTION 6.2
DESIGN FOR SHEAR

This section applies to stiffened or un-stiffened webs of singly or doubly symmetric beams, including hybrid beams, and channels subject to shear in the plane of the web. For shear in the weak direction of the shapes above, pipes, and unsymmetric sections, see Section 8.2. For web panels subject to high shear, see Section 11.1.7. For shear strength at connections, see Sections 10.4 and 10.5.

6.2.1 Web Area Determination

The web area A_w shall be taken as the overall depth d times the web thickness t_w.

6.2.2 Design Shear Strength

The design shear strength of stiffened or un-stiffened webs is $\phi_v V_n$,

where

ϕ_v = 0.90

V_n = nominal shear strength defined as follows.

(a) For $h/t_w \leq 1.1\sqrt{k_v E/F_{yw}}$:

$$V_n = 0.6F_{yw}A_w \qquad \textbf{(6.2-1)}$$

(b) For $1.10\sqrt{k_v E/F_{yw}} < h/t_w \leq 1.37\sqrt{k_v E/F_{yw}}$:

$$V_n = 0.6F_{yw}A_w(1.10\sqrt{k_v E/F_{yw}})/(h/t_w) \qquad \textbf{(6.2-2)}$$

(c) For $h/t_w > 1.37\sqrt{k_v E/F_{yw}}$:

$$V_n = A_w(0.91Ek_v)/(h/t_w)^2 \qquad \textbf{(6.2-3)}$$

where

$k_v = 5 + 5/(a/h^2)$

= 5 when $a/h > 3$ or $a/h > [260/(h/t)]^2$

a = distance between transverse stiffeners, mm

h = for rolled shapes, the clear distance between flanges less the fillet or corner radius, mm

= for built-up welded sections, the clear distance between flanges, mm

= for built-up bolted or riveted sections, the distance between fastener lines, mm

6.2.3 **Transverse Stiffeners.** Transverse stiffeners are not needed where the required shear, V_u, as determined by structural analysis for the factored loads, is less than or equal to $0.6\phi_v A_w F_{yw} C_v$, where $\phi_v = 0.90$ and the shear coefficient C_v, defined in Section 7.3, is determined for $k_v = 5$.

Transverse stiffeners used to develop the web design shear strength as provided in Section 6.2.2 shall have a moment of inertia about an axis in the web center for stiffener pairs, or about the face in contact with the web plate for single stiffeners, which shall not be less than $at_w^3 j$, where

$$j = 2.5/(a/h)^2 - 2 \geq 0.5 \qquad \textbf{(6.2-4)}$$

Intermediate stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which intermediate stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe of the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange. When lateral bracing is attached to a stiffener, or a pair of stiffeners, these, in turn, shall be connected to the compression flange to transmit one percent of the total flange stress, unless the flange is composed only of angles. Bolts connecting stiffeners to the girder web shall be spaced not more than 300 mm on center. If intermittent fillet welds are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 250 mm.

SECTION 6.3
WEB-TAPERED MEMBERS

The design of tapered members meeting the requirements of this section shall be governed by the provisions of Chapters 4 through 8, except as modified by the following provisions:

6.3.1 **General Requirements.** In order to qualify under this specification, a tapered member shall meet the following requirements:

(1) It shall possess at least one axis of symmetry, which shall be perpendicular to the plane of bending if moments are present.

(2) The flanges shall be of equal and constant area.

(3) The depth shall vary linearly as

$$d = d_o\left(1 + \gamma \frac{z}{L}\right) \qquad \textbf{(6.3-1)}$$

where

γ = $(d_L - d_o)/d_o \leq$ the smaller of $0.268(L/d_o)$ or 6.0

d_o = depth at smaller end of member, mm

d_L = depth at larger end of member, mm

z = distance from the smaller end of member, mm

L = unbraced length of member measured between the center of gravity of the bracing members, mm

6.3.2 **Design Tensile Strength.** The design strength of tapered tension members shall be determined in accordance with Section 4.1.

6.3.3 **Design Compressive Strength.** The design strength of tapered compression members shall be determined in accordance with Section 5.3, using an effective slenderness parameter λ_{eff} computed as follows:

$$\lambda_{eff} = \frac{S}{\pi}\sqrt{\frac{QF_y}{E}} \qquad \textbf{(6.3-2)}$$

where

S = KL/r_{oy} for weak axis buckling and $K_\gamma L/r_{ox}$ for strong axis buckling

K = effective length factor for a prismatic member

K_γ = effective length factor for a tapered member as determined by a rational analysis

r_{ox} = strong axis radius of gyration at the smaller end of a tapered member, mm

r_{oy} = weak axis radius of gyration at the smaller end of a tapered member, mm

F_y = specified minimum yield stress, MPa

Q = reduction factor

= 1.0 if all elements meet the limiting width-thickness ratios λ_r of Section 2.5.1

= Q_sQ_a, determined in accordance with Section 2.5.3, if any stiffened and/or unstiffened elements exceed the ratios λ_r of Section 2.5.1

E = modulus of elasticity for steel, MPa

The smallest area of the tapered member shall be used for A_g in Equation 5.2-1.

6.3.4 **Design Flexural Strength.** The design flexural strength of tapered flexural members for the limit state of lateral-torsional buckling is $\phi_b M_n$, where $\phi_b = 0.90$ and the nominal strength is

$$M_n = (5/3)S'_x F_{b\gamma} \qquad \textbf{(6.3-3)}$$

where

S'_x = the section modulus of the critical section of the unbraced beam length under consideration

$$F_{b\gamma} = \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{wr}}}\right]F_y \le 0.60F_y \qquad \textbf{(6.3-4)}$$

Unless $F_{b\gamma} \le F_y / 3$, in which case

$$F_{b\gamma} = B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \qquad \textbf{(6.3-5)}$$

In the preceding equations,

$$F_{s\gamma} = \frac{0.41E}{h_s L d_o / A_f} \qquad \textbf{(6.3-6)}$$

$$F_{w\gamma} = \frac{5.9E}{(h_w L / r_{To})^2} \qquad \textbf{(6.3-7)}$$

where

h_s = factor equal $1.0 + 0.230\gamma\sqrt{Ld_o / A_f}$

h_w = factor equal to $1.0 + 0.00385\gamma\sqrt{L / r_{To}}$

r_{To} = radius of gyration of a section at the smaller end, considering only the compression flange plus one-third of the compression web area, taken about an axis in the plane of the web, mm

A_f = area of the compression flange, mm^2

and where B is determined as follows:

(a) When the maximum moment M_2 in three adjacent segments of approximately equal unbraced length is located within the central segment and M_1 is the larger moment at one end of the three-segment portion of a member:

$$B = 1.0 + 0.37\left(1.0 + \frac{M_1}{M_2}\right) + 0.50\gamma\left(1.0\frac{M_1}{M_2}\right) \geq 1.0 \qquad \textbf{(6.3-8)}$$

(b) When the largest computed bending stress f_{b2} occurs at the larger end of two adjacent segments of approximately equal unbraced lengths and f_{b1} is the computed bending stress at the smaller end of the two-segment portion of a member:

$$B = 1.0 + 0.58\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) - 0.70\gamma\left(1.0\frac{f_{b1}}{f_{b2}}\right) \geq 1.0 \qquad \textbf{(6.3-9)}$$

(c) When the largest computed bending stress f_{b2} occurs at the smaller end of two adjacent segments of approximately equal unbraced length and f_{b1} is the computed bending stress at the larger end of the two-segment portion of a member:

$$B = 1.0 + 0.55\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) + 2.20\gamma\left(1.0\frac{f_{b1}}{f_{b2}}\right) \geq 1.0 \qquad \textbf{(6.3-10)}$$

In the foregoing, $\gamma = (d_L - d_o)/d_o$ is calculated for the unbraced length that contains the maximum computed bending stress. M_1/M_2 is considered as negative when producing single curvature. In the rare case where M_1/M_2 is positive, it is recommended that it be taken as zero. f_{b1}/f_{b2} is considered as negative when producing single curvature. If a point of contraflexure occurs in one of two adjacent unbraced segments, f_{b1}/f_{b2} is considered as positive. The ratio $f_{b1}/f_{b2} \neq 0$.

(d) When the computed bending stress at the smaller end of a tapered member or segment thereof is equal to zero:

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}} \qquad \textbf{(6.3-11)}$$

where $\gamma = (d_L - d_o)/d_o$ is calculated for the unbraced length adjacent to the point of zero bending stress.

6.3.5 **Design Shear Strength.** The design shear strength of tapered flexural members shall be determined in accordance with Section 6.2.

6.3.6 **Combined Flexure and Axial Force.** For tapered members with a single web taper subject to compression and bending about the major axis, Equation 8.1-1 applies, with the following modifications. P_n and P_{ex} shall be determined for the properties of the smaller end, using appropriate effective length factors. M_{nx}, M_u, and M_{px} shall be determined for the larger end; $M_{nx} = (5/3)S'_x F_{b\gamma}$, where S'_x is the elastic section modulus of the larger end, and $F_{b\gamma}$ is the design flexural stress of tapered members. C_{mx} is replaced by C'_m determined as follows:

(a) When the member is subjected to end moments which cause single curvature bending and approximately equal computed moments at the ends:

$$C'_m = 1.0 + 0.1\left(\frac{P_u}{\phi_b P_{ex}}\right) + 0.3\left(\frac{P_u}{\phi_b P_{ex}}\right)^2 \qquad \textbf{(6.3-12)}$$

(b) When the computed bending moment at the smaller end of the unbraced length is equal to zero:

$$C'_m = 1.0 + 0.9\left(\frac{P_u}{\phi_b P_{ex}}\right) + 0.6\left(\frac{P_u}{\phi_b P_{ex}}\right)^2 \qquad \textbf{(6.3-13)}$$

When the effective slenderness parameter $\lambda_{eff} \geq 1.5$ and combined stress is checked incrementally along the length, the actual area and the actual section modulus at the section under investigation is permitted to be used.

SECTION 6.4
BEAMS AND GIRDERS WITH WEB OPENINGS

The effect of all web openings on the design strength of steel and composite beams shall be determined. Adequate reinforcement shall be provided when the required strength exceeds the design strength of the member at the opening.

CHAPTER 7
PLATE GIRDERS

I-shaped plate girders shall be distinguished from I-shaped beams on the basis of the web slenderness ratio h / t_w. When this value is greater than λ_r the provisions of Sections 7.1 and 7.2 shall apply for design flexural strength. For $h / t_w \leq \lambda_r$, the provisions of Chapter 6 shall apply for design flexural strength. For girders with unequal flanges, see Section 2.5.1.

The design shear strength and transverse stiffener design shall be based on either Section 6.2 (without tension-field action) or Section 7.3 (with tension-field action). For girders with unequal flanges, see Section 2.5.1.

SECTION 7.1
LIMITATIONS

Doubly and singly symmetric single-web non-hybrid and hybrid plate girders loaded in the plane of the web shall be proportioned according to the provisions of this section or Section 6.2, provided that the following limits are satisfied:

(a) For $\frac{a}{h} \leq 1.5$

$$\frac{h}{t_w} \leq 11.7 \frac{E}{F_{yf}} \qquad \textbf{(7.1-1)}$$

(b) For $\frac{a}{h} > 1.5$:

$$\left(\frac{h}{t_w} \leq \frac{0.48E}{\sqrt{F_{yf}\left(F_{yf} + 114\right)}} \right) \qquad \textbf{(7.1-2)}$$

where

a = clear distance between transverse stiffeners, mm

h = clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, mm

t_w = web thickness, mm

F_{yf} = specified minimum yield stress of a flange, MPa

In unstiffened girders h/t_w shall not exceed 260.

SECTION 7.2
DESIGN FLEXURAL STRENGTH

The design flexural strength for plate girders with slender webs shall be $\phi_b M_n$, where $\phi_b = 0.90$ and M_n is the lower value obtained according to the limit states of tension-flange yield and compression-flange buckling. For girders with unequal flanges, see Section 2.5.1 for the determination of λ_r for the limit state of web local buckling.

(a) For tension-flange yield:

$$M_n = S_{xt} R_e F_{yt} \qquad \textbf{(7.2-1)}$$

(b) For compression-flange buckling:

$$M_n = S_{xc} R_{PG} R_e F_{cr} \quad \textbf{(7.2-2)}$$

where

$$R_{PG} = 1 - \frac{a_r}{1{,}200 + 300a_r}\left(\frac{h_c}{t_w} - 5.70\sqrt{\frac{E}{F_{cr}}}\right) \le 1.0 \quad \textbf{(7.2-3)}$$

R_e = hybrid girder factor

= $\dfrac{12 + a_r(3m - m^3)}{12 + 2a_r} \le 1.0$ (for non-hybrid girders, R_e = 1.0)

a_r = ratio of web area to compression flange area (≤ 10)

m = ratio of web yield stress to flange yield stress or to F_{cr}

F_{cr} = critical compression flange stress, MPa

F_{yt} = yield stress of tension flange, MPa

S_{xc} = section modulus referred to compression flange, mm^3

S_{xt} = section modulus referred to tension flange, mm^3

h_c = twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside of the face of the compression flange when welds are used, mm

The critical stress F_{cr} to be used is dependent upon the slenderness parameters λ, λ_p, λ_r, and C_{PG} as follows:

(a) For $\lambda \le \lambda_p$

$$F_{cr} = F_{yf} \quad \textbf{(7.2-4)}$$

(b) For $\lambda_p < \lambda \le \lambda_r$:

$$F_{cr} = C_b F_{yf}\left[1 - \frac{1}{2}\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \le F_{yf} \quad \textbf{(7.2-5)}$$

(c) For $\lambda > \lambda_r$:

$$F_{cr} = \frac{C_{PG}}{\lambda^2} \quad \textbf{(7.2-6)}$$

In the foregoing, the slenderness parameter shall be determined for both the limit state of lateral-torsional buckling and the limit state of flange local buckling; the slenderness parameter which results in the lowest value of F_{cr} governs.

(a) For the limit state of lateral-torsional buckling:

$$\lambda = \frac{L_b}{r_T} \quad \textbf{(7.2-7)}$$

$$\lambda_p = 1.76\sqrt{\frac{E}{F_{yf}}} \quad \textbf{(7.2-8)}$$

$$\lambda_r = 4.44\sqrt{\frac{E}{F_{yf}}} \quad \textbf{(7.2-9)}$$

$$C_{PG} = 1970000\ C_b \quad \textbf{(7.2-10)}$$

where

C_b = see Section 6.1.2, Equation 6.1-3

r_T = radius of gyration of compression flange plus one-third of the compression portion of the web, mm

(b) For the limit state of flange local buckling:

$$\lambda = \frac{b_f}{2t_f} \quad \textbf{(7.2-11)}$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_{yf}}} \quad \textbf{(7.2-12)}$$

$$\lambda_r = 1.35\sqrt{\frac{E}{F_{yf}/k_c}} \quad \textbf{(7.2-13)}$$

$$C_{PG} = 180650k_c \quad \textbf{(7.2-14)}$$

$$C_b = 1.0$$

where $k_c = 4/\sqrt{h/t_w}$ and $0.35 \leq k_c \leq 0.763$

The limit state of flexural web local buckling is not applicable.

SECTION 7.3
DESIGN SHEAR STRENGTH

The design shear strength with tension field action shall be $\phi_n V_n$, kN,
where
ϕ_v = 0.90 and V_n is determined as follows:

(a) For $h/t_w \leq 1.10\sqrt{k_v E/F_{yw}}$:

$$V_n = 0.6F_{yw}A_w \quad \textbf{(7.3-1)}$$

(b) For $h/t_w > 1.10\sqrt{k_v E/F_{yw}}$:

$$V_n = 0.6F_{yw}A_w\left(C_v + \frac{1-C_v}{1.15\sqrt{1+(a/h)^2}}\right) \quad \textbf{(7.3-2)}$$

Also see Sections 7.4 and 7.5.

Tension field action is not permitted for end-panels in non-hybrid plate girders, all panels in hybrid and web-tapered plate girders, and when a / h exceeds 3.0 or [260 (h / t_w)]. For these cases, the nominal strength is:

$$V_n = 0.6F_{yw}A_wC_v \quad \textbf{(7.3-3)}$$

The web plate buckling coefficient k_v is given as

$$k_v = 5 + \frac{5}{(a/h)^2} \quad \textbf{(7.3-4)}$$

except that k_v shall be taken as 5.0 if a / h exceeds 3.0 or $[260 / (h / t_w)]^2$.

The shear coefficient C_v is determined as follows:

(a) For $1.10\sqrt{\dfrac{k_v E}{F_{yw}}} \le \dfrac{h}{t_w} \le 1.37\sqrt{\dfrac{k_v E}{F_{yw}}}$:

$$C_v = \frac{1.10\sqrt{k_v E / F_{yw}}}{h / t_w} \quad \textbf{(7.3-5)}$$

(b) For $\dfrac{h}{t_w} > 1.37\sqrt{\dfrac{k_v E}{F_{yw}}}$:

$$C_v = \frac{1.51 k_v E}{(h / t_w)^2 F_{yw}} \quad \textbf{(7.3-6)}$$

SECTION 7.4
TRANSVERSE STIFFENERS

Transverse stiffeners are not required in plate girders where $h / t_w \le 2.45\sqrt{E / F_{yw}}$, or where the required shear V_u, as determined by structural analysis for the factored loads, is less than or equal to $0.60\phi_v F_{yw} A_w C_v$ where C_v is determined for $k_v = 5$ and $\phi_v = 0.90$. Stiffeners may be required in certain portions of a plate girder to develop the required shear or to satisfy the limitations given in Section 7.1. Transverse stiffeners shall satisfy the requirements of Section 6.2.3.

When designing for tension field action, the stiffener area A_{st} shall not be less than

$$\frac{F_{yw}}{F_{yst}}\left[0.15 D h t_w (1 - C_v)\frac{V_u}{\phi V_n} - 18 t_w^2\right] \ge 0 \quad \textbf{(7.4-1)}$$

where

F_{yst}	=	specified yield stress of the stiffener material, MPa
D	=	1 for stiffeners in pairs
	=	1.8 for single angle stiffeners
	=	2.4 for single plate stiffeners
C_v and V_n	=	are defined in Section 7.3, and V_u is the required shear at the location of the stiffener.

SECTION 7.5
FLEXURE-SHEAR INTERACTION

For $0.6\phi V_n \le V_u \le \phi V_n$ and $0.75\phi M_n \le M_u \le \phi M_n$, plate girders with webs designed for tension field action shall satisfy the additional flexure-shear interaction criterion:

$$\frac{M_u}{\phi M_n} + 0.625\frac{V_u}{\phi V_n} \le 1.375 \quad \textbf{(7.5-1)}$$

where

M_n	=	nominal flexural strength of plate girder from Section 7.2 or Section 6.1
ϕ	=	0.90
V_n	=	nominal shear strength from Section 7.3

CHAPTER 8
MEMBERS UNDER COMBINED FORCES AND TORSION

This chapter applies to prismatic members subject to axial force and flexure about one or both axes of symmetry, with or without torsion, and torsion only. For web-tapered members, see Section 6.3.

SECTION 8.1
SYMMETRIC MEMBERS SUBJECT TO BENDING AND AXIAL FORCE

8.1.1 **Doubly and Singly Symmetric Members in Flexure and Tension**. The interaction of flexure and tension in symmetric shapes shall be limited by Equations 8.1-1a and 8.1-1b.

(a) For $\frac{P_u}{\phi P_n} \geq 0.2$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad \textbf{(8.1-1a)}$$

(b) For $\frac{P_u}{\phi P_n} < 0.2$

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad \textbf{(8.1-1b)}$$

where

P_u = required tensile strength, N

P_n = nominal tensile strength determined in accordance with Section 4.1, (N)

M_u = required flexural strength determined in accordance with Section 3.1, (N-mm)

M_n = nominal flexural strength determined in accordance with Section 6.1, (N-mm)

x = subscript relating symbol to strong axis bending

y = subscript relating symbol to weak axis bending

ϕ = ϕ_t = resistance factor for tension (see Section 4.1)

ϕ_b = resistance factor for flexure = 0.90

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equations 8.1-1a and 8.1-1b.

8.1.2 **Doubly and Singly Symmetric Members in Flexure and Compression.** The interaction of flexure and compression in symmetric shapes shall be limited by Equations 8.1-1a and 8.1-1b.

where

P_u = required compressive strength, N

P_n = nominal compressive strength determined in accordance with Section 5.2, N

$\phi = \phi_c$ = resistance factor for compression = 0.85 (see Section 5.2)

ϕ_b = resistance factor for flexure = 0.90

SECTION 8.2
UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

The design strength, ϕF_n, of the member shall equal or exceed the required strength expressed in terms of the normal stress f_{un} or the shear stress f_{uv}, determined by elastic analysis for the factored loads:

(a) For the limit state of yielding under normal stress:

$$f_{un} \le \phi F_y \quad \textbf{(8.2-1)}$$

$$\phi = 0.90$$

$$F_n = F_y$$

(b) For the limit state of yielding under shear stress:

$$f_{uv} \le 0.6\phi F_n \quad \textbf{(8.2-2)}$$

$$\phi = 0.90$$

$$F_n = F_y$$

(c) For the limit state of buckling:

$$f_{un} \text{ or } f_{uv} \le \phi_c F_n, \text{ as applicable} \quad \textbf{(8.2-3)}$$

$$\phi_c = 0.85$$

$$F_n = F_{cr}$$

Some constrained local yielding is permitted adjacent to areas which remain elastic.

SECTION 8.3
ALTERNATIVE INTERACTION EQUATIONS FOR MEMBERS UNDER COMBINED STRESS

This section provides alternative interaction equations for braced frames with I-shaped members with $b_f / d \le 1.0$ and box-shaped members.

For I-shaped members with $b_f / d \le 1.0$ and box-shaped members, the use of the following interaction equations in lieu of Equations 8.1-1a and 8.1-1b is permitted for braced frames only. Both Equations 8.3-1 and 8.3-2 shall be satisfied.

$$\left(\frac{M_{ux}}{\phi_b M'_{px}}\right)^{\zeta} + \left(\frac{M_{uy}}{\phi_b M'_{py}}\right)^{\zeta} \le 1.0 \quad \textbf{(8.3-1)}$$

$$\left(\frac{C_{mx} M_{ux}}{\phi_b M'_{nx}}\right)^{\eta} + \left(\frac{C_{my} M_{uy}}{\phi_b M'_{ny}}\right)^{\eta} \le 1.0 \quad \textbf{(8.3-2)}$$

The terms in Equations 8.3-1 and 8.3-2 are determined as follows:

(a) For I-shaped members:

For $b_f / d < 0.5$:

$$\zeta = 1.0$$

For $0.5 \le b_f / d \le 1.0$:

$$\zeta = 1.6 - \frac{P_u / P_y}{2\left[\ln(P_u / P_y)\right]} \quad \textbf{(8.3-3)}$$

For $b_f / d < 0.3$:

$$\eta = 1.0$$

For $0.3 \le b_f / d \le 1.0$:

$$\eta = 0.4 + \frac{P_u}{P_y} + \frac{b_f}{d} \ge 1.0 \quad \textbf{(8.3-4)}$$

where

b_f = flange width, (mm)

d = member depth, (mm)

C_m = coefficient applied to the bending term in the interaction equation for prismatic members and dependent on column curvature caused by applied moments, see Section 3.1.

$$M'_{px} = 1.2M_{px}\left[1-(P_u / P_y)\right] \le M_{px} \quad \textbf{(8.3-5)}$$

$$M'_{py} = 1.2M_{py}\left[1-(P_u / P_y)^2\right] \le M_{py} \quad \textbf{(8.3-6)}$$

$$M'_{nx} = M_{nx}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ex}}\right) \quad \textbf{(8.3-7)}$$

$$M'_{ny} = M_{ny}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ey}}\right) \quad \textbf{(8.3-8)}$$

(b) For box-section members:

$$\zeta = 1.7 - \frac{P_u / P_y}{\ln(P_u / P_y)} \quad \textbf{(8.3-9)}$$

$$\eta = 1.7 - \frac{P_u / P_y}{\ln(P_u / P_y)} - a\lambda_x\left(\frac{P_u}{P_y}\right)^b > 1.1 \quad \textbf{(8.3-10)}$$

For $P_u / P_y \le 4.0$, $a = 0.06$, and $b = 1.0$;

For $P_u / P_y > 4.0$, $a = 0.15$, and $b = 2.0$:

$$M'_{px} = 1.2M_{px}\left[1-P_u / P_y\right] \le M_{px} \quad \textbf{(8.3-11a)}$$

$$M'_{py} = 1.2M_{py}\left[1-P_u / P_y\right] \le M_{py} \quad \textbf{(8.3-11b)}$$

$$M'_{nx} = M_{nx}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ex}}\frac{1.25}{(B/H)^{1/3}}\right) \qquad \textbf{(8.3-12)}$$

$$M'_{ny} = M_{ny}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ey}}\frac{1.25}{(B/H)^{1/2}}\right) \qquad \textbf{(8.3-13)}$$

where

P_n = nominal compressive strength determined in accordance with Section 5.2, N

P_u = required axial strength, N

P_y = compressive yield strength $A_g F_y$, N

ϕ_b = resistance factor for flexure = 0.90

ϕ_c = resistance factor for compression = 0.85

P_e = Euler buckling strength $A_g F_y / \lambda_c^2$, where λ_c is the column slenderness parameter defined by Equation 5.2-4, N

M_u = required flexural strength, N-mm

M_n = nominal flexural strength, determined in accordance with Section 6.1, N-mm

M_p = plastic moment $\leq 1.5 F_y S$, N-mm

λ_x = column slenderness parameter with respect to the strong axis

B = outside width of box section parallel to major principal axis x, mm

H = outside depth of box section perpendicular to major principal axis x, mm

CHAPTER 9
COMPOSITE MEMBERS

This chapter applies to composite columns composed of rolled or built-up structural steel shapes, pipe or HSS, and structural concrete acting together and to steel beams supporting a reinforced concrete slab so interconnected that the beams and the slab act together to resist bending. Simple and continuous composite beams with shear connectors and concrete-encased beams, constructed with or without temporary shores, are included.

SECTION 9.1
DESIGN ASSUMPTIONS AND DEFINITIONS

Force Determination. In determining forces in members and connections of a structure that includes composite beams, consideration shall be given to the effective sections at the time each increment of load is applied.

Elastic Analysis. For an elastic analysis of continuous composite beams without haunched ends, it is permissible to assume that the stiffness of a beam is uniform throughout the beam length. The stiffness is permitted to be computed using the weighted average of the moments of inertia in the positive moment region and the negative moment region.

Plastic Analysis. When plastic analysis is used, as limited in Section 1.5.1, the strength of flexural composite members shall be determined from plastic stress distributions.

Plastic Stress Distribution for Positive Moment. If the slab in the positive moment region is connected to the steel beam with shear connectors, a concrete stress of $0.85f_c^{'}$ is permitted to be assumed uniformly distributed throughout the effective compression zone, where $f_c^{'}$ is the specified compressive strength of the concrete. Concrete tensile strength shall be neglected. A uniformly distributed steel stress of F_y shall be assumed throughout the tension zone and throughout the compression zone in the structural steel section. The net tensile force in the steel section shall be equal to the compressive force in the concrete slab.

Plastic Stress Distribution for Negative Moment. If the slab in the negative moment region is connected to the steel beam with shear connectors, a tensile stress of F_{yr} shall be assumed in all adequately developed longitudinal reinforcing bars within the effective width of the concrete slab. Concrete tensile strength shall be neglected. A uniformly distributed steel stress of F_y shall be assumed throughout the tension zone and throughout the compression zone in the structural steel section. The net compressive force in the steel section shall be equal to the total tensile force in the reinforcing steel.

Elastic Stress Distribution. When a determination of elastic stress distribution is required, strains in steel and concrete shall be assumed directly proportional to the distance from the neutral axis. The stress shall equal strain times modulus of elasticity for steel, E, or modulus of elasticity for concrete, E_c. Concrete tensile strength shall be neglected. Maximum stress in the steel shall not exceed F_y. Maximum compressive stress in the concrete shall not exceed $0.85f_c^{'}$. In composite hybrid beams, the maximum stress in the steel flange shall not exceed

F_{yf} but the strain in the web may exceed the yield strain; the stress shall be taken as F_{yw} at such locations.

Fully Composite Beam. Shear connectors are provided in sufficient numbers to develop the maximum flexural strength of the composite beam. For elastic stress distribution it shall be assumed that no slip occurs.

Partially Composite Beam. The shear strength of shear connectors governs the flexural strength of the partially composite beam. Elastic computations such as those for deflections, fatigue, and vibrations shall include the effect of slip.

Concrete-Encased Beam. A beam totally encased in concrete cast integrally with the slab may be assumed to be interconnected to the concrete by natural bond, without additional anchorage, provided that: (1) concrete cover over the beam sides and soffit is at least 50 mm; (2) the top of the beam is at least 38 mm below the top and 50 mm above the bottom of the slab; and (3) concrete encasement contains adequate mesh or other reinforcing steel to prevent spalling of concrete.

Composite Column. A steel column fabricated from rolled or built-up steel shapes and encased in structural concrete or fabricated from steel pipe or HSS and filled with structural concrete.

Encased Composite Column. A steel column fabricated from rolled or built-up shapes and encased in structural concrete.

Filled Composite Column. Structural steel HSS or pipes that are filled with structural concrete.

SECTION 9.2
COMPRESSION MEMBERS

9.2.1 **Limitations.** To qualify as a composite column, the following limitations shall be met:

(1) The cross-sectional area of the steel shape, pipe, or HSS shall comprise at least four percent of the total composite cross section.

(2) Concrete encasement of a steel core shall be reinforced with longitudinal load-carrying bars, longitudinal bars to restrain concrete, and lateral ties. Longitudinal load-carrying bars shall be continuous at framed levels; longitudinal restraining bars may be interrupted at framed levels. The spacing of ties shall be not greater than two-thirds of the least dimension of the composite cross section. The cross-sectional area of the transverse and longitudinal reinforcement shall be at least 180 mm^2 per m of bar spacing. The encasement shall provide at least 38 mm of clear cover outside of both transverse and longitudinal reinforcement.

(3) Concrete shall have a specified compressive strength f_c' of not less than 21 MPa nor more than 55 MPa for normal weight concrete and not less than 28 MPa for lightweight concrete.

(4) The specified minimum yield stress of structural steel and reinforcing bars used in calculating the strength of a composite column shall not exceed 415 MPa.

(5) The minimum wall thickness of structural steel pipe or HSS filled with concrete shall be equal to $b\sqrt{F_y / 3E}$ for each face of width b in rectangular sections and $D\sqrt{F_y / 8E}$ for circular sections of outside diameter D.

9.2.2 **Design Strength.** The design strength of axially loaded composite columns is $\phi_c P_n$,

where

ϕ_c = 0.85

P_n = nominal axial compressive strength determined from Equations 5.2-1 through 5.2-4 with the following modifications:

(1) A_g is replaced by A_s, the gross area of steel shape, pipe, or HSS, mm^2

(2) r is replaced by r_m, the radius of gyration of the steel shape, pipe, or HSS except that for steel shapes it shall not be less than 0.3 times the overall thickness of the composite cross section in the plane of buckling, mm

(3) F_y is replaced by F_{my}, the modified yield stress from Equation 9.2-1

$$F_{my} = F_y + c_1 F_{yr} (A_r / A_s) + c_2 f_c' (A_c / A_s) \qquad \textbf{(9.2-1)}$$

(4) E is replaced by E_m, the modified modulus of elasticity from Equation 9.2-2.

$$E_m = E + c_3 E_c (A_c / A_s) \qquad \textbf{(9.2-2)}$$

where

A_c = area of concrete, mm^2

A_r = area of longitudinal reinforcing bars, mm^2

A_s = area of steel, mm^2

E = modulus of elasticity of steel, MPa

E_c = modulus of elasticity of concrete. E_c is permitted to be computed from $E_c = 0.041 w^{1.5} \sqrt{f_c'}$ where w, the unit weight of concrete, is expressed in kg/m^3 and f_c' is expressed in MPa.

F_y = specified minimum yield stress of steel shape, pipe, or HSS, MPa

F_{yr} = specified minimum yield stress of longitudinal reinforcing bars, MPa

f_c' = specified compressive strength of concrete, MPa

c_1, c_2, c_3 = numerical coefficients. For concrete-filled pipe and HSS: $c_1 = 1.0$, $c_2 = 0.85$, and $c_3 = 0.4$; for concrete-encased shapes, $c_1 = 0.7$, $c_2 = 0.6$, and $c_3 = 0.2$

9.2.3 **Columns with Multiple Steel Shapes.** If the composite cross section includes two or more steel shapes, the shapes shall be interconnected with lacing, tie plates, or batten plates to prevent buckling of individual shapes before hardening of concrete.

9.2.4 **Load Transfer.** Loads applied to axially loaded encased composite columns shall be transferred between the steel and concrete in accordance with the following requirements:

(a) When the external force is applied directly to the steel section, shear connectors shall be provided to transfer the force V_u' as follows:

$$V_u' = V_u(1 - A_s F_y / P_n) \quad \textbf{(9.2-3)}$$

where

V_u = force introduced to column, N

A_s = area of steel section, mm^2

F_y = yield strength of the steel section, MPa

P_n = nominal compressive strength of the composite column without consideration of slenderness effects, N

(b) When the external force is applied directly to the concrete encasement, shear connectors shall be provided to transfer the force V_u' as follows:

$$V_u' = V_u(A_s F_y / P_n) \quad \textbf{(9.2-4)}$$

Shear connectors transferring the force V_u' shall be distributed along the length of the member. The maximum connector spacing shall be 405 mm and connectors shall be placed on at least two faces of the steel shape in a configuration symmetrical about the steel shape axes.

Where the supporting concrete area in direct bearing is wider than the loaded area on one or more sides and otherwise restrained laterally on the remaining sides, the maximum design strength shall be:

$$\phi_B\, 1.7 f_c'\, A_B \quad \textbf{(9.2-5)}$$

where

ϕ_B = 0.65

A_B = loaded area, mm^2

SECTION 9.3
FLEXURAL MEMBERS

9.3.1 **Effective Width.** The effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline, each of which shall not exceed:

(1) one-eighth of the beam span, center-to-center of supports;

(2) one-half the distance to the centerline of the adjacent beam; or

(3) the distance to the edge of the slab.

9.3.2 **Design Strength of Beams with Shear Connectors.** The positive design flexural strength $\phi_b M_n$ shall be determined as follows:

(a) For $h/t_w \le 3.76\sqrt{E/F_{yf}}$:

$\phi_b = 0.85$; M_n shall be determined from the plastic stress distribution on the composite section.

(b) For $h/t_w > 3.76\sqrt{E/F_{yf}}$:

$\phi_b = 0.90$; M_n shall be determined from the superposition of elastic stresses, considering the effects of shoring.

The negative design flexural strength $\phi_b M_n$ shall be determined for the steel section alone, in accordance with the requirements of Chapter 6.

Alternatively, the negative design flexural strength $\phi_b M_n$ shall be computed with $\phi_b = 0.85$ and M_n determined from the plastic stress distribution on the composite section, provided that:

(1) Steel beam is an adequately braced compact section, as defined in Section 2.5.

(2) Shear connectors connect the slab to the steel beam in the negative moment region.

(3) Slab reinforcement parallel to the steel beam, within the effective width of the slab, is properly developed.

9.3.3 **Design Strength of Concrete-Encased Beams.** The design flexural strength $\phi_b M_n$ shall be computed with $\phi_b = 0.90$ and M_n determined from the superposition of elastic stresses, considering the effects of shoring.

Alternatively, the design flexural strength $\phi_b M_n$ shall be computed with $\phi_b = 0.90$ and M_n determined from the plastic stress distribution on the steel section alone.

If shear connectors are provided and the concrete meets the requirements of Section 9.2.1(2), the design flexural strength $\phi_b M_n$ shall be computed based upon the plastic stress distribution on the composite section with $\phi_b = 0.85$.

9.3.4 **Strength During Construction.** When temporary shores are not used during construction, the steel section alone shall have adequate strength to support all loads applied prior to the concrete attaining 75 percent of its specified strength $f_c^{'}$.

The design flexural strength of the steel section shall be determined in accordance with the requirements of Section 6.1.

9.3.5 Formed Steel Deck

9.3.5.1 **General.** The design flexural strength, $\phi_b M_n$, of composite construction consisting of concrete slabs on formed steel deck connected to steel beams shall be determined by the applicable portions of Section 9.3.2, with the following modifications:

(1) This section is applicable to decks with nominal rib height not greater than 75 mm. The average width of concrete rib or haunch w_r shall be not less than 50 mm, but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck.

(2) The concrete slab shall be connected to the steel beam with welded stud shear connectors 19 mm or less in diameter (AWS D1.1). Studs shall be welded either through the deck or directly to the steel beam. Stud shear connectors, after installation, shall extend not less 38 mm above the top of the steel deck.

The slab thickness above the steel deck shall be not less than 50 mm.

9.3.5.2 **Deck Ribs Oriented Perpendicular to Steel Beam.** Concrete below the top of the steel deck shall be neglected in determining section properties and in calculating A_c for deck ribs oriented perpendicular to the steel beams.

The spacing of stud shear connectors along the length of a supporting beam shall not exceed 915 mm.

The nominal strength of a stud shear connector shall be the value stipulated in Section 9.5 multiplied by the following reduction factor:

$$\frac{0.85}{\sqrt{N_r}}(w_r / h_r)[(H_s / h_r)-1.0] \le 1.0 \qquad \textbf{(9.3-1)}$$

where

h_r = nominal rib height, mm

H_s = length of stud connector after welding, mm, not to exceed the value $h_r + 75$ mm in computations, although actual length may be greater

N_r = number of stud connectors in one rib at a beam intersection, not to exceed three in computations, although more than three studs may be installed

w_r = average width of concrete rib or haunch (as defined in Section 9.3.5.1), mm

Where there is only a single stud placed in a rib oriented perpendicular to the steel beam, the reduction factor of Equation 9.3-1 shall not exceed 0.75.

To resist uplift, steel deck shall be anchored to all supporting members at a spacing not to exceed 460 mm. Such anchorage shall be provided by stud connectors, a combination of stud connectors and arc spot (puddle) welds, or other devices specified by the designer.

9.3.5.3 **Deck Ribs Oriented Parallel to Steel Beam.** Concrete below the top of the steel deck may be included in determining section properties and shall be included in calculating A_c in Section 9.5.

Steel deck ribs over supporting beams may be split longitudinally and separated to form a concrete haunch.

When the nominal depth of steel deck is 38 mm or greater, the average width w_r of the supported haunch or rib shall be not less than 50 mm for the first stud in the transverse row plus four stud diameters for each additional stud.

The nominal strength of a stud shear connector shall be the value stipulated in Section 9.5, except that when w_r / h_r is less than 1.5, the value from Section 9.5 shall be multiplied by the following reduction factor:

$$0.6(w_r / h_r)[(H_s / h_r)-1.0] \le 1.0 \qquad \textbf{(9.3-2)}$$

where h_r and H_s are as defined in Section 9.3.5.2 and w_r is the average width of concrete rib or haunch as defined in Section 9.3.5.1.

9.3.6 **Design Shear Strength.** The design shear strength of composite beams shall be determined by the shear strength of the steel web, in accordance with Section 6.2.

SECTION 9.4
COMBINED COMPRESSION AND FLEXURE

The interaction of axial compression and flexure in the plane of symmetry on composite members shall be limited by Section 8.1.2 with the following modifications:

M_n = nominal flexural strength determined from plastic stress distribution on the composite cross section except as provided below, N-mm

P_{e1}, P_{e2} = $A_s F_{my} / \lambda_c^2$ elastic buckling load, N

F_{my} = modified yield stress, MPa, see Section 9.2

ϕ_b = resistance factor for flexure from Section 9.3

ϕ_c = resistance factor for compression = 0.85

λ_c = column slenderness parameter defined by Equation 5.2-4 as modified in Section 9.2.2

When the axial term in Equations 8.1-1a and 8.1-1b is less than 0.3, the nominal flexural strength M_n shall be determined by straight line transition between the nominal flexural strength determined from the plastic distribution on the composite cross sections at $(P_u/\phi_c P_n) = 0.3$ and the flexural strength at $P_u = 0$ as determined in Section 9.3. If shear connectors are used at $P_u = 0$, they shall be provided whenever $P_u/\phi_c P_n$ is less than 0.3.

SECTION 9.5
SHEAR CONNECTORS

This section applies to the design of stud and channel shear connectors. For connectors of other types, see Section 9.6.

9.5.1 **Materials.** Shear connectors shall be headed steel studs not less than four stud diameters in length after installation, or hot rolled steel channels. The stud connectors shall conform to the requirements of Section 1.3.6. The channel connectors shall conform to the requirements of Section 1.3. Shear connectors shall be embedded in concrete slabs made with ASTM C33 aggregate or with rotary kiln produced aggregates conforming to ASTM C330, with concrete unit weight not less than 1440 kg/m^3.

9.5.2 **Horizontal Shear Force.** The entire horizontal shear at the interface between the steel beam and the concrete slab shall be assumed to be transferred by shear connectors, except for concrete-encased beams as defined in Section 9.1. For composite action with concrete subject to flexural compression, the total horizontal shear force between the point of maximum positive moment and the point of zero moment shall be taken as the smallest of the following: (a) $0.85 f_c' A_c$; (b) $A_s F_y$; and (c) ΣQ_n;

where

A_c = area of concrete slab within effective width, mm^2

A_s = area of steel cross section, mm^2

ΣQ_n = sum of nominal strengths of shear connectors between the point of maximum positive moment and the point of zero moment, N

For hybrid beams, the yield force shall be computed separately for each component of the cross section; $A_s F_y$ of the entire cross section is the sum of the component yield forces.

In continuous composite beams where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel beam, the total horizontal shear force between the point of maximum negative moment and

the point of zero moment shall be taken as the smaller of $A_r F_{yr}$ and ΣQ_n;

where

A_r = area of adequately developed longitudinal reinforcing steel within the effective width of the concrete slab, mm^2

F_{yr} = minimum specified yield stress of the reinforcing steel, MPa

9.5.3 **Strength of Stud Shear Connectors.** The nominal strength of one stud shear connector embedded in a solid concrete slab is

$$Q_n = 0.5A_{sc}\sqrt{f_c' E_c} \leq A_{sc}F_u \qquad \textbf{(9.5-1)}$$

where

A_{sc} = cross-sectional area of stud shear connector, mm^2

F_u = specified minimum tensile strength of a stud shear connector, MPa

E_c = modulus of elasticity of concrete, MPa

For a stud shear connector embedded in a slab on a formed steel deck, refer to Section 9.3 for reduction factors given by Equations 9.3-1 and 9.3-2 as applicable. The reduction factors apply only to the $0.5A_{sc}\sqrt{f_c' E_c}$ term in Equation 9.5-1.

9.5.4 **Strength of Channel Shear Connectors.** The nominal strength of one channel shear connector embedded in a solid concrete slab is

$$Q_n = 0.3(t_f + 0.5t_w)L_c\sqrt{f_c' E_c} \qquad \textbf{(9.5-2)}$$

where

t_f = flange thickness of channel shear connector, mm

t_w = web thickness of channel shear connector, mm

L_c = length of channel shear connector, mm

9.5.5 **Required Number of Shear Connectors.** The number of shear connectors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment shall be equal to the horizontal shear force as determined in Section 9.5.2 divided by the nominal strength of one shear connector as determined from Section 9.5.3 or Section 9.5.4.

9.5.6 **Shear Connector Placement and Spacing.** Shear connectors required on each side of the point of maximum bending moment, positive or negative, shall be distributed uniformly between that point and the adjacent points of zero moment, unless otherwise specified. However, the number of shear connectors placed between any concentrated load and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the concentrated load point.

Shear connectors shall have at least 25 mm of lateral concrete cover, except for connectors installed in the ribs of formed steel decks. The diameter of studs shall not be greater than 2.5 times the thickness of the flange to which they are welded, unless located over the web. The minimum center-to-center spacing of stud connectors shall be six diameters along the longitudinal axis of the supporting composite beam and four diameters transverse to the longitudinal axis of the supporting composite beam, except that within the ribs of formed steel decks oriented perpendicular to the steel beam the minimum center-to-center spacing

shall be four diameters in any direction. The maximum center-to-center spacing of shear connectors shall not exceed eight times the total slab thickness. Also see Section 9.3.5.2.

SECTION 9.6
SPECIAL CASES

When composite construction does not conform to the requirements of Section 9.1 through Section 9.5, the strength of shear connectors and details of construction shall be established by a suitable test program.

CHAPTER 10
CONNECTIONS, JOINTS, AND FASTENERS

This chapter applies to connecting elements, connectors, and the affected elements of the connected members subject to static loads. For connections subject to fatigue, see Section 11.3.

SECTION 10.1
GENERAL PROVISIONS

10.1.1 **Design Basis.** Connections consist of affected elements of connected members (e.g., beam webs), connecting elements (e.g., gussets, angles, brackets), and connectors (e.g., welds, bolts, rivets). These components shall be proportioned so that their design strength equals or exceeds the required strength determined by structural analysis for factored loads acting on the structure or a specified proportion of the strength of the connected members, whichever is appropriate.

10.1.2 **Simple Connections.** Connections of beams, girders, or trusses shall be designed as flexible and are permitted to be proportioned for the reaction shears only, except as otherwise indicated in the design documents. Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, some inelastic but self-limiting deformation in the connection is permitted.

10.1.3 **Moment Connections.** End connections of restrained beams, girders, and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connections.

10.1.4 **Compression Members with Bearing Joints.** When columns bear on bearing plates or are finished to bear at splices, there shall be sufficient connectors to hold all parts securely in place.
When compression members other than columns are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportioned for 50 % of the required strength of the member.
All compression joints shall be proportioned to resist any tension developed by the factored load combinations stipulated in Section 1.4.

10.1.5 **Splices in Heavy Sections.** This paragraph applies to ASTM A6/A6M Group 4 and 5 and equivalent rolled shapes, or shapes built-up by welding plates more than 50 mm thick together to form the cross section, and where the cross section is to be spliced and subject to primary tensile stresses due to tension or flexure. When the individual elements of the cross section are spliced prior to being joined to form the cross section in accordance with AWS D1.1, Article 5.21.6, the applicable provisions of AWS D1.1 apply in lieu of the requirements of this section. When tensile forces in these sections are to be transmitted through splices by complete-joint-penetration groove welds, material notch-toughness requirements as given in Section 1.3.1.3, weld access hole details as given in Section 10.1.6, welding preheat requirements as given in Section 10.2.8, and thermal-cut surface preparation and inspection requirements as given in Section 12.2.2 apply.

At tension splices in ASTM A6/A6M Group 4 and 5 and equivalent shapes and built-up members of material more than 50 mm thick, weld tabs and backing shall be removed and the surfaces ground smooth.

When splicing ASTM A6/A6M Group 4 and 5 and equivalent rolled shapes and their equivalents or shapes built-up by welding plates more than 50 mm thick to form a cross section, and where the section is to be used as a primary compression member, all weld access holes required to facilitate groove welding operations shall satisfy the provisions of Section 10.1.6.

Alternatively, splicing of such members subject to compression, including members which are subject to tension due to wind or seismic loads, shall be accomplished using splice details which do not induce large weld shrinkage strains; for example partial-joint-penetration flange groove welds with fillet-welded surface lap plate splices on the web, bolted lap plate splices, or combination bolted/fillet-welded lap plate splices.

10.1.6 **Beam Copes and Weld Access Holes.** All weld access holes required to facilitate welding operations shall have a length from the toe of the weld preparation not less than 1.5 times the thickness of the material in which the hole is made. The height of the access hole shall be adequate for deposition of sound weld metal in the adjacent plates and provide clearance for weld tabs for the weld in the material in which the hole is made, but not less than the thickness of the material. In hot-rolled shapes and built-up shapes, all beam copes and weld access holes shall be shaped free of notches and sharp re-entrant corners, except that when fillet web-to-flange welds are used in built-up shapes, access holes are permitted to terminate perpendicular to the flange.

For ASTM A6/A6M Group 4 and 5 and equivalent shapes and built-up shapes of material more than 50 mm thick, the thermally cut surfaces of beam copes and weld access holes shall be ground to bright metal and inspected by either magnetic particle or dye penetrant methods prior to deposition of splice welds. If the curved transition portion of weld access holes and beam copes are formed by predrilled or sawed holes, that portion of the access hole or cope need not be ground. Weld access holes and beam copes in other shapes need not be ground nor inspected by dye penetrant or magnetic particle methods.

10.1.7 **Minimum Strength of Connections.** Connections providing design strength shall be designed to support a factored load not less than 44 kN, except for lacing, sag rods, or girts.

10.1.8 **Placement of Welds and Bolts.** Groups of welds or bolts at the ends of any member which transmit axial force into that member shall be sized so that the center of gravity of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of statically-loaded single angle, double angle, and similar members.

10.1.9 **Bolts in Combination with Welds.** In new work, A307 bolts or high-strength bolts proportioned as bearing-type connections shall not be considered as sharing the load in combination with welds. Welds, if used, shall be proportioned for the entire force in the connection. In slip-critical connections, high-strength bolts are permitted to be considered as sharing the load with the welds. These calculations shall be made at factored loads.

In making welded alterations to structures, existing rivets and high-strength bolts tightened to the requirements for slip-critical connections are permitted to be utilized for carrying loads present at the time of alteration and the welding need only provide the additional design strength required.

10.1.10 High-Strength Bolts in Combination with Rivets. In both new work and alterations, in connections designed as slip-critical connections in accordance with the provisions of Section 10.3, high-strength bolts are permitted to be considered as sharing the load with rivets.

10.1.11 Limitations on Bolted and Welded Connections. Fully pretensioned high-strength bolts (see Table 10.3-1) or welds shall be used for the following connections:

Column splices in all tier structures 60 m or more in height.

Column splices in tier structures 30 m to 60 m in height, if the least horizontal dimension is less than 40 percent of the height.

Column splices in tier structures less than 30 m in height, if the least horizontal dimension is less than 25 percent of the height.

Connections of all beams and girders to columns and of any other beams and girders on which the bracing of columns is dependent, in structures over 38 m in height.

In all structures carrying cranes of over 50 kN capacity: roof-truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports.

Connections for supports of running machinery, or of other live loads which produce impact or reversal of stress.

Any other connections stipulated on the design drawings.

In all other cases connections are permitted to be made with A307 bolts or snug-tight high-strength bolts.

For the purpose of this section, the height of a tier structure shall be taken as the vertical distance from the curb level to the highest point of the roof beams in the case of flat roofs, or to the mean height of the gable in the case of roofs having a slope of more than 25 percent. Where the curb level has not been established, or where the structure does not adjoin a street, the mean level of the adjoining land shall be used instead of curb level. It is permissible to exclude penthouses in computing the height of the structure.

SECTION 10.2
WELDS

All provisions of AWS D1.1, apply under this code, except the provisions applicable to Tubular Structures, which are outside the scope of SBC 306, and except that the provisions of the listed SBC 306 sections in lieu of the cited AWS Code provisions as follows:

Section 10.1.5 and 10.1.6 in lieu of AWS D1.1 Section 5.17.
Section 10.2.2 in lieu of AWS D1.1 Section 2.4.1.1.
Table 10.2-5 in lieu of AWS D1.1 Table 2.3
Table 11.3-1 in lieu of AWS D1.1 Section 2.27.1
Section 11.3 in lieu of AWS Section 2, Part C
Section 13.2.2 in lieu of AWS Sections 5.15.1.2, 5.15.4.3 and 5.15.4.4

The length and disposition of welds, including end returns shall be indicated on the design and shop drawings.

10.2.1 Groove Welds

10.2.1.1 Effective Area. The effective area of groove welds shall be considered as the effective length of the welds times the effective throat thickness.

The effective length of a groove weld shall be the width of the part joined.

The effective throat thickness of a complete-joint-penetration groove weld shall be the thickness of the thinner part joined.

The effective throat thickness of a partial-joint-penetration groove weld shall be as shown in Table 10.2-1.

The effective throat thickness of a flare groove weld when flush to the surface of a bar or 90° bend in formed section shall be as shown in Table 10.2-2. Random sections of production welds for each welding procedure, or such test sections as may be required by design documents, shall be used to verify that the effective throat is consistently obtained.

Larger effective throat thicknesses than those in Table 10.2-2 are permitted, provided the fabricator can establish by qualification the consistent production of such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication or as required by the designer.

10.2.1.2 Limitations. The minimum effective throat thickness of a partial-joint-penetration groove weld shall be as shown in Table 10.2-3. Weld size is determined by the thicker of the two parts joined, except that the weld size need not exceed the thickness of the thinnest part joined, even when a larger size is required by calculated strength. For this exception, particular care shall be taken to provide sufficient preheat for soundness of the weld.

TABLE 10.2-1
Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

<table>
<tr><th>Welding Process</th><th>Welding Position</th><th>Included Angle at Root of Groove</th><th>Effective Throat Thickness</th></tr>
<tr><td>Shielded metal arc
Submerged arc</td><td rowspan="4">All</td><td>J or U joint</td><td rowspan="3">Depth of chamfer</td></tr>
<tr><td>Gas metal arc</td><td rowspan="2">Bevel or V joint $\geq 60^{o}$</td></tr>
<tr><td rowspan="2">Flux-cored arc</td></tr>
<tr><td>Bevel or V joint < 60^{o}
Bevel or V but $\geq 45°$</td><td>Depth of chamfer
Minus 3 mm</td></tr>
</table>

TABLE 10.2-2
Effective Throat Thickness of Flare Groove Welds

Type of Weld	Radius (R) of Bar or Bend	Effective Throat Thickness
Flare bevel groove	All	$5/16R$
Flare V-groove	All	$1/2R$

[a] Use $3/8R$ for Gas Metal Arc Welding (except short circuiting transfer process) when $R \geq 25$ mm

TABLE 10.2-3 Minimum Effective Throat Thickness of Partial-Joint-Penetration Groove Welds	
Material Thickness of Thicker Part Joined (mm)	**Minimum Effective Throat Thickness[a], (mm)**
To 6 inclusive	3
Over 6 to 13	5
Over 13 to 19	6
Over 19 to 38	8
Over 38 to 57	10
Over 57 to 150	13
Over 150	16
[a] See Table 10.2-1	

10.2.2 Fillet Welds

10.2.2.1 Effective Area. The effective area of fillet welds shall be as defined in AWS D1.1 Section 2.4.3 and 2.11. The effective throat thickness of a fillet weld shall be the shortest distance from the root of the joint to the face of the diagrammatic weld, except that for fillet welds made by the submerged arc process, the effective throat thickness shall be taken equal to the leg size for 10 mm and smaller fillet welds, and equal to the theoretical throat plus 3 mm for fillet welds over 10 mm.

For fillet welds in holes and slots, the effective length shall be the length of the centerline of the weld along the center of the plane through the throat. In the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot, in the plane of the faying surface.

TABLE 10.2-4 Minimum Size of Fillet Welds	
Material Thickness of Thicker Part Joined, mm	**Minimum Size of Fillet Weld[a] mm**
To 6 inclusive	3
Over 6 to 13	5
Over 13 to 19	6
Over 19	8
[a] Leg dimension of fillet welds. Single pass welds must be used. [b] See Section 10.2.2b for maximum size of fillet welds.	

10.2.2.2 Limitations

The *minimum size of fillet welds* shall be not less than the size required to transmit calculated forces nor the size as shown in Table 10.2-4, which is based upon experiences and provides some margin for uncalculated stress encountered during fabrication, handling, transportation, and erection. These provisions do not apply to fillet weld reinforcements of partial- or complete-joint-penetration welds.

The *maximum size of fillet welds* of connected parts shall be:

(a) Along edges of material less than 6 mm thick, not greater than the thickness of the material.

(b) Along edges of material 6 mm or more in thickness, not greater than the thickness of the material minus 2 mm, unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld is permitted to be less than 2 mm provided the weld size is clearly verifiable.

For flange-web welds and similar connections, the actual weld size need not be larger than that required to develop the web capacity, and the requirements of Table

10.2-4 need not apply.

The *minimum effective length of fillet welds* designed on the basis of strength shall be not less than four times the nominal size, or else the size of the weld shall be considered not to exceed ¼ of its effective length. If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. For the effect of longitudinal fillet weld length in end connections upon the effective area of the connected member, see Section 2.3.

For end-loaded fillet welds with a length up to 100 times the leg dimension, it is permitted to take the effective length equal to the actual length. When the length of the end-loaded fillet weld exceeds 100 times the weld size, the effective length shall be determined by multiplying the actual length by the reduction factor, β.

where

$$\beta = 1.2 - 0.002(L/w) \leq 1.0 \qquad \textbf{(10.2-1)}$$

L = actual length of end-loaded weld, mm

w = weld leg size, mm

When the length of the weld exceeds 300 times the leg size, the value of β shall be taken as 0.60.

Intermittent fillet welds may be used to transfer calculated stress across a joint or faying surfaces when the strength required is less than that developed by a continuous fillet weld of the smallest permitted size, and to join components of built-up members. The effective length of any segment of intermittent fillet welding shall be not less than four times the weld size, with a minimum of 38 mm.

In lap joints, the minimum amount of lap shall be five times the thickness of the thinner part joined, but not less than 25 mm. Lap joints joining plates or bars subjected to axial stress that utilize transverse fillet welds only shall be fillet welded along the end of both lapped parts, except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

Fillet weld terminations are permitted to extend to the ends or sides of parts or be stopped short or boxed except as limited by the following:

(1) For lap joints in which one part extends beyond an edge subject to calculated tensile stress, fillet welds shall terminate not less than the size of the weld from that edge.

(2) For connections and structural elements with cyclic forces, normal to outstanding legs, of frequency and magnitude that would tend to cause a progressive fatigue failure initiating from a point of maximum stress at the end of the weld, fillet welds shall be returned around the corner for a distance not less than the smaller of two times the weld size or the width of the part.

(3) For connections whose design requires flexibility of the outstanding legs, if end returns are used, their length shall not exceed four times the nominal size of the weld.

(4) Fillet welds joining transverse stiffeners to plate girder webs shall end not less than four times nor more than six times the thickness of the web from the web toe of the web-to-flange welds, except where the ends of stiffeners are welded to the flange.

(5) Fillet welds, which occur on opposite sides of a common plane, shall be interrupted at the corner common to both welds.

Fillet welds in holes or slots may be used to transmit shear in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds may overlap, subject to the provisions of Section 10.2. Fillet welds in holes or slots are not to be considered plug or slot welds.

10.2.3 Plug and Slot Welds

10.2.3.1 Effective Area. The effective shearing area of plug and slot welds shall be considered as the nominal cross-sectional area of the hole or slot in the plane of the faying surface

10.2.3.2 Limitations. Plug or slot welds are permitted to be used to transmit shear in lap joints or to prevent buckling of lapped parts and to join component parts of built-up members.

The diameter of the holes for a plug weld shall not be less than the thickness of the part containing it plus 8 mm, rounded to the next larger even mm, nor greater than the minimum diameter plus 3 mm or 2.25 times the thickness of the weld. The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.

The length of slot for a slot weld shall not exceed 10 times the thickness of the weld. The width of the slot shall be not less than the thickness of the part containing it plus 8 mm rounded to the next larger even mm, nor shall it be larger than 2.25 times the thickness of the weld. The ends of the slot shall be semicircular or shall have the corners rounded to a radius of not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the width of the slot. The minimum center-to-center spacing in a longitudinal direction on any line shall be two times the length of the slot.

The thickness of plug or slot welds in material 16 mm or less in thickness shall be equal to the thickness of the material. In material over 16 mm thick, the thickness of the weld shall be at least one-half the thickness of the material but not less than 16 mm.

10.2.4 Design Strength. The design strength of welds shall be the lower value of (a) $\phi F_{BM} A_{BM}$ and (b) $\phi F_w A_w$, when applicable. The values of ϕ, F_{BM}, and F_w and limitations thereon are given in Table 10.2-5,

where

F_{BM} = nominal strength of the base material, MPa

F_w = nominal strength of the weld electrode, MPa

A_{BM} = cross-sectional area of the base material, mm^2

A_w = effective cross-sectional area of the weld, mm^2

ϕ = resistance factor

Alternatively, in lieu of the constant design strength for fillet welds given in Table 10.2-5, fillet welds loaded in-plane are permitted to be designed in accordance with the following procedure.

(a) For a linear weld group loaded in-plane through the center of gravity, the design strength is $\phi F_w A_w$,

where

$$F_w = 0.60\, F_{EXX}(1.0 + 0.50 \sin^{1.5}\theta) \qquad \textbf{(10.2-2)}$$

ϕ = 0.75

F_{EXX} = electrode classification number, i.e., minimum specified strength, MPa

θ = angle of loading measured from the weld longitudinal axis, degrees

A_w = effective area of weld throat, mm^2.

(b) For weld elements within a weld group that are loaded in-plane and analyzed using an instantaneous center of rotation method, the components of the design strength are $\phi\ F_{wx}\ A_w$ and $\phi\ F_{wy}\ A_w$

Where:

$F_{wx} = \Sigma F_{wix}$

$F_{wy} = \Sigma F_{wiy}$

$F_{wi} = 0.60\, F_{EXX}\,(1.0 + 0.50 \sin^{1.5}\theta)\, f(p)$

$f(p) = [p(1.9 - 0.9p)]^{0.3}$

$\phi = 0.75$

where

F_{wi} = nominal stress in any ith weld element, MPa

F_{wix} = x component of stress F_{wi}

F_{wiy} = y component of stress F_{wi}

p = Δ_i / Δ_m, ratio of element i deformation to its deformation at maximum stress

Δ_m = $0.209(\theta + 2)^{-0.32}\, w$, deformation of weld element at maximum stress, mm

Δ_i = deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i, mm.

= $r_i\ \Delta_u / r_{crit}$

$\Delta_{\underline{u}}$ = $1.087(\theta + 6)^{-0.65} w \leq 0.17w$, deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, mm

w = leg size of the fillet weld, mm

r_{crit} = distance from instantaneous center of rotation to weld element with minimum $\Delta_{\underline{u}} / r_i$ ratio, mm

10.2.5 **Combination of Welds.** If two or more of the general types of welds (groove, fillet, plug, slot) are combined in a single joint, the design strength of each shall be separately computed with reference to the axis of the group in order to determine the design strength of the combination.

TABLE 10.2-5
Design Strength of Welds

Types of Weld and Stress [a]	Material	Resistance Factor ~	Nominal Strength F_{BM} or Fw	Filler Metal Requirements [b, c]
Complete-Joint-Penetration Groove Weld				
Tension normal to Effective area	Base	0.90	F_y	Matching filler metal shall be used. For CVN requirements see footnote [d].
Compression normal to effective area	Base	0.90	F_y	Filler metal with a strength level equal to or less than matching filler metal is permitted to be used.
Tension or compression parallel to axis of weld				
Shear on effective Area	Base Weld	0.90 0.80	$0.60F_y$ $0.60F_{EXX}$	
Partial-Joint-Penetration Groove Weld				
Compression normal to effective area	Base	0.90	F_y	Filler metal with a strength level equal to or less than matching filler metal is permitted to be used.
Tension or compression parallel to axis of weld [e]				
Shear parallel to axis of weld	Base Weld	[f] 0.75	[f] $0.60F_{EXX}$	
Tension normal to Effective area	Base Weld	0.90 0.80	F_y $0.60F_{EXX}$	
Fillet Welds				
Shear on effective Area	Base Weld	[f] 0.75	[f] $0.60F_{EXX}$ [g]	Filler metal with a strength level equal to or less than
Tension or compression parallel to axis of weld [e]	Base	0.90	F_y	matching filler metal is permitted to be used.
Plug or Slot Welds				
Shear parallel to faying surfaces (on Effective area)	Base Weld	[f] 0.75	[f] $0.60F_{EXX}$	Filler metal with a strength level equal to or less than matching filler metal is permitted to be used.

[a] For definition of effective area, see Section 10.2.
[b] For matching filler metal, see Table 3.1, AWS D1.1.
[c] Filler metal one strength level stronger than matching filler metal is permitted.
[d] For T and corner joints with the backing bar left in place during service, filler metal with a classification requiring a minimum Charpy V-notch (CVN) toughness of 27 J @ 4°C shall be used. If filler metal without the required toughness is used and the backing bar is left in place, the joint shall be sized using the resistance factor and nominal strength for a partial-joint-penetration weld.
[e] Fillet welds and partial-joint-penetration groove welds joining component elements of built-up members, such as flange-to-web connections, are not required to be designed with Regard to the tensile or compressive stress in these elements parallel to the axis of the welds.
[f] The design of connected material is governed by Sections 10.4 and 10.5.
[g] For alternative design strength, see below.

10.2.6 **Weld Metal Requirements.** The choice of electrode for use with complete-joint-penetration groove welds subject to tension normal to the effective area shall comply with the requirements for matching weld metals given in AWS D1.1.

Weld metal with a specified Charpy V-notch (CVN) toughness of 27 J at 4°C shall be used in the following joints:

(a) Complete-joint-penetration groove welded T and corner joints with steel backing left in place, subject to tension normal to the effective area, unless the joints are designed as noted in Table 10.2-5 (see footnote d).

(b) Complete-joint-penetration groove welded splices subject to tension normal to the effective area in Group 4 and Group 5 shapes and shapes built up by welding plates more than 50 mm thick.

The manufacturer's Certificate of Conformance shall be sufficient evidence of compliance.

10.2.7 **Mixed Weld Metal.** When notch-toughness is specified, the process consumables for all weld metal, tack welds, root pass, and subsequent passes deposited in a joint shall be compatible to assure notch-tough composite weld metal.

10.2.8 **Preheat for Heavy Shapes.** For ASTM A6/A6M Group 4 and 5 and equivalent shapes and welded built-up members made of plates more than 50 mm thick, a preheat equal to or greater than 350°F (175°C) shall be used when making groove-weld splices.

SECTION 10.3
BOLTS AND THREADED PARTS

10.3.1 **High-Strength Bolts.** Use of high-strength bolts shall conform to the provisions of the *Load and Resistance Factor Design specification for Structural Joints Using ASTM A325 or A490 Bolts,* as approved by the Research Council on Structural Connections, except as otherwise provided in this code.

If required to be tightened to more than 50 percent of their specified minimum tensile strength, A449 bolts in tension and bearing-type shear connections shall have an ASTM F436 hardened washer installed under the bolt head, and the nuts shall meet the requirements of ASTM A563. When assembled, all joint surfaces, including those adjacent to the washers, shall be free of scale, except tight mill scale. All A325 or A325M and A490 or A490M bolts shall be tightened to a bolt tension not less than that given in Table 10.3-1, except as noted below. Tightening shall be done by any of the following methods: turn-of-nut method, a direct tension indicator, calibrated wrench, or alternative design bolt.

TABLE 10.3-1
Minimum Bolt Pretension, kN*

Bolt Size, mm	A325M Bolts	A490M Bolts
M16	91	114
M20	142	179
M22	176	221
M24	205	257
M27	267	334
M30	326	408
M36	475	595

*Equal to 0.70 of minimum tensile strength of bolts, rounded off to nearest kN, as specified in ASTM specifications for A325M and A490M bolts with UNC threads.

Bolts need only be tightened to the snug-tight condition when in: (a) bearing-type connections where slip is permitted, or (b) tension or combined shear and tension applications, for ASTM A325 or A325M bolts only, where loosening or fatigue due to vibration or load fluctuations are not design considerations. The snug-tight condition is defined as the tightness attained by either a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench that brings the connected plies into firm contact. The nominal strength value given in Table 10.3-2 and Table 10.3-5 shall be used for bolts tightened to the snug-tight condition. Bolts tightened only to the snug-tight condition shall be clearly identified on the design and erection drawings.

When A490 or A490M bolts over 25 mm in diameter are used in slotted or oversize holes in external plies, a single hardened washer conforming to ASTM F436, except with 8 mm minimum thickness, shall be used in lieu of the standard washer. In slip-critical connections in which the direction of loading is toward an edge of a connected part, adequate design bearing strength shall be provided based upon the applicable requirements of Section 10.3.10.

TABLE 10.3-2
Design Strength of Fasteners

Description of Fasteners	Tensile Strength		Shear Strength in Bearing-type Connections	
	Resistance Factor ϕ	Nominal Strength, MPa	Resistance Factor ϕ	Nominal Strength, MPa
A307 bolts	0.75	310 [a]	0.75	165 [b,e]
A325 or A325M bolts, when threads are not excluded from shear planes		620 [d]		330 [e]
A325 or A325M bolts, when threads are excluded from shear planes		620 [d]		414 [e]
A490 or A490M bolts, when threads are not excluded from shear planes		780 [d]		414 [e]
A490 or A490M bolts, when threads are excluded from shear planes		780 [d]		520 [e]
Threaded parts meeting the requirements of Section A3, when threads are not excluded from shear planes		$0.75F_u$ [a,c]		$0.40F_u$
Threaded parts meeting the requirements of Section A3, when threads are excluded from shear planes		$0.75F_u$ [a,c]		$0.50F_u$ [a,c]
A502, Gr. 1, hot-driven Rivets		310 [a]		172 [e]
A502, Gr. 2 & 3, hot-driven Rivets		414 [a]		228 [e]

[a] Static loading only.

[b] Threads permitted in shear planes.

[c] The nominal tensile strength of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter, A_D shall be larger than the nominal body area of the rod before upsetting times F_y.

[d] For A325 or A325M and A490 or A490M bolts subject to tensile fatigue loading, see Section 11.3.

[e] When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 1270 mm, tabulated values shall be reduced by 20 percent.

10.3.2 Size and Use of Holes. The *maximum sizes* of holes for rivets and bolts are given in Table 10.3-3, except that larger holes, required for tolerance on location of anchor rods in concrete foundations, are allowed in column base details.

Standard holes shall be provided in member-to-member connections, unless over-sized, short-slotted, or long-slotted holes in bolted connections are approved by the designer. Finger shims up to 6 mm are permitted in slip-critical connections designed on the basis of standard holes without reducing the nominal shear strength of the fastener to that specified for slotted holes.

Oversized holes are allowed in any or all plies of slip-critical connections, but they shall not be used in bearing-type connections. Hardened washers shall be installed over oversized holes in an outer ply.

TABLE 10.3-3
Nominal Hole Dimensions, mm

Bolt Diameter	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-slot (Width x Length)	Long-slot (Width x Length)
M16	18	20	18 × 22	18 × 40
M20	22	24	22 × 26	22 × 50
M22	24	28	24 × 30	24 × 55
M24	27 [a]	30	27 × 32	27 × 60
M27	30	35	30 × 37	30 × 67
M30	33	38	33 × 40	33 × 75
≥ M36	$d + 3$	$d + 8$	$(d + 3) \times (d + 10)$	$(d + 3) \times 2.5d$

[a] clearance provided allows the use of a 25 mm, bolt, if desirable.

Short-slotted holes are allowed in any or all plies of slip-critical or bearing-type connections. The slots are permitted to be used without regard to direction of loading in slip-critical connections, but the length shall be normal to the direction of the load in bearing-type connections. Washers shall be installed over short-slotted holes in an outer ply; when high-strength bolts are used, such washers shall be hardened.

Long-slotted holes are allowed in only one of the connected parts of either a slip-critical or bearing-type connection at an individual faying surface. Long-slotted holes are permitted to be used without regard to direction of loading in slip-critical connections, but shall be normal to the direction of load in bearing-type connections. Where long-slotted holes are used in an outer ply, plate washers, or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than 8 mm thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

10.3.3 Minimum Spacing. The distance between centers of standard, oversized, or slotted holes, shall not be less than 2.67 times the nominal diameter of the fastener; a distance of $3d$ is preferred. Refer to Section 10.3.10 for bearing strength requirements.

10.3.4 **Minimum Edge Distance.** The distance from the center of a standard hole to an edge of a connected part shall not be less than either the applicable value from Table 10.3-4, or as required in Section 10.3.10. The distance from the center of an oversized or slotted hole to an edge of a connected part shall be not less than that required for a standard hole to an edge of a connected part plus the applicable increment C_2 from Table 10.3-6. Refer to Section 10.3.10 for bearing strength requirements.

TABLE 10.3-4
Minimum Edge Distance,[a] mm , From Center of Standard Hole[b] to Edge of Connected Part

Nominal Rivet or Bolt Diameter (mm)	At Sheared Edges	At Rolled Edges of Plates, Gas Cut Edges[c]
16	28	22
20	34	26
22	38[d]	28
24	42[d]	30
27	48	34
30	52	38
36	64	46
Over 36	1.75d	1.25d

[a] Lesser edge distances are permitted to be used provided Equations from Section 10.3.10, as appropriate, are satisfied.

[b] For oversized or slotted holes, see Table 10.3-6.

[c] All edge distances in this column are permitted to be reduced 3 mm when the hole is at a point where stress does not exceed 25 percent of the maximum design strength in the element.

[d] These are permitted to be 32 mm at the ends of beam connection angles and shear end plates.

10.3.5 **Maximum Spacing and Edge Distance.** The maximum distance from the center of any bolt or rivet to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed 150 mm. The longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates shall be as follows:

(a) or painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner plate or 305 mm.

(b) For unpainted members of weathering steel subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner plate or 180 mm.

TABLE 10.3-5
Nominal Tension Stress *(F_t)*, MPa
Fasteners in Bearing-type Connections

Description of Fasteners	Threads Included in the Shear Plane	Threads Excluded from the Shear Plane
A307 bolts	$407 - 2.5f_v \leq 310$	
A325M bolts	$807 - 2.5f_v \leq 621$	$807 - 2.0f_v \leq 621$
A490M bolts	$1010 - 2.5f_v \leq 779$	$1010 - 2.0f_v \leq 779$
Threaded parts A449 bolts Over 38 mm diameter	$0.98F_u - 2.5f_v \leq 0.75F_u$	$0.98F_u - 2.0f_v \leq 0.75F_u$
A502 Gr. 1 rivets	$407 - 2.4f_v \leq 310$	
A502 Gr. 2 rivets	$538 - 2.4f_v \leq 414$	

TABLE 10.3-5A Nominal Tension Stress (F_t), MPa Fasteners in Bearing-type Connections		
Description of Fasteners	**Threads Included in the Shear Plane**	**Threads Excluded from the Shear Plane**
A307 bolts	$\sqrt{310^2 - 6.25 f_v^2}$	
A325M bolts	$\sqrt{621^2 - 6.25 f_v^2}$	$\sqrt{621^2 - 4.00 f_v^2}$
A490M bolts	$\sqrt{779^2 - 6.31 f_v^2}$	$\sqrt{779^2 - 4.04 f_v^2}$
Threaded parts A449 bolts over 38 mm	$\sqrt{(0.75F_u)^2 - 6.25 f_v^2}$	$\sqrt{(0.75F_u)^2 - 4.00 f_v^2}$
A502 Gr. 1 rivets	$\sqrt{310^2 - 5.76 f_v^2}$	
A502 Gr. 2 rivets	$\sqrt{414^2 - 5.86 f_v^2}$	

10.3.6 **Design Tension or Shear Strength.** The design tension or shear strength of a high-strength bolt or threaded part is

$$\phi F_n A_b,$$

where

ϕ = resistance factor tabulated in Table 10.3-2

F_n = nominal tensile strength F_t, or shear strength, F_v, tabulated in Table 10.3-2, MPa

A_b = nominal unthreaded body area of bolt or threaded part (for upset rods, see Footnote c, Table 10.3-2), mm^2

The applied load shall be the sum of the factored loads and any tension resulting from prying action produced by deformation of the connected parts.

10.3.7 **Combined Tension and Shear in Bearing-Type Connections.** The design strength of a bolt or rivet subject to combined tension and shear is

$$\phi F_v A_b,$$

where

ϕ = 0.75

F_t = nominal tension stress computed from the equations in Table 10.3-5 as a function of f_v, the required shear stress produced by the factored loads. Alternately, the use of the equations in Table 10.3-5A is permitted. The design shear strength ϕF_v tabulated in Table 10.3-2 shall equal or exceed the shear stress, f_v.

10.3.8 **High-Strength Bolts in Slip-Critical Connections.** The design for shear of high-strength bolts in slip-critical connections shall be in accordance with Section 10.3.8 and checked for shear in accordance with Sections 10.3.6 and 10.3.7 and bearing in accordance with Sections 10.3.1 and 10.3.10.

10.3.8.1 **Slip-Critical Connections Designed at Factored Loads.** The design slip resistance per bolt, ϕr_{str}, shall equal or exceed the required force per bolt due to factored loads,

$$r_{str} = 1.13\ \mu_{\mu} T_b N_s \qquad \textbf{(10.3-1)}$$

where:

T_b = minimum fastener tension given in Table 10.3-1, kN

N_s = number of slip planes

μ = mean slip coefficient for Class A, B, or C surfaces, as applicable, or as established by tests

(a) For Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel),

$$\mu = 0.33$$

(b) For Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel),

$$\mu = 0.50$$

(c) For Class C surfaces (hot-dip galvanized and roughened surfaces),

$$\mu = 0.35$$

ϕ = resistance factor

(a) For standard holes, $\phi = 1.0$

(b) For oversized and short-slotted holes, $\phi = 0.85$

(c) For long-slotted holes transverse to the direction of load, $\phi = 0.70$

(d) For long-slotted holes parallel to the direction of load, $\phi = 0.60$

Finger shims up to 6 mm are permitted to be introduced into slip-critical connections designed on the basis of standard holes without reducing the design shear stress of the fastener to that specified for slotted holes.

TABLE 10.3-6
Values of Edge Distance Increment C_2, mm

Nominal Diameter of Fastener (mm)	Oversized Holes	Slotted Holes		
		Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots [a]	
< 22	2	3	0.75d	0
24	3	3		
≥ 27	3	5		

[a] When length of slot is less than maximum allowable (see Table 10.3-5), C_2 are permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

10.3.9 **Combined Tension and Shear in Slip-Critical Connections.** The design of slip-critical connections subject to tensile forces shall be in accordance with either Sections 10.3.9.1 and 10.3.8.1 or Sections 10.3.9.2 and 10.3.8.2.

10.3.9.1 **Slip-Critical Connections Designed at Factored Loads.** When a slip-critical connection is subjected to an applied tension T_u that reduces the net clamping force, the slip resistance ϕr_{str} according to Section 10.3.8.1, shall be multiplied by the following factor:

$$1 - \{ T_u / (1.13\ T_b N_b)\}$$

where

T_b = minimum bolt pretension from Table 10.3-1, kN

N_b = number of bolts carrying factored-load tension T_u

10.3.10 **Bearing Strength at Bolt Holes.** Bearing strength shall be checked for both bearing-type and slip-critical connections. The use of oversized holes and short- and long-slotted holes parallel to the line of force is restricted to slip-critical connections per Section 10.3.2. The design bearing strength at bolt holes is ϕR_n,

where

$\phi = 0.75$

and R_n is determined as follows:

(a) For a bolt in a connection with standard, oversized, and short-slotted holes in-dependent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force:

when deformation at the bolt hole at service load is a design consideration:

$$R_n = 1.2L\,tF_u \leq 2.4\,dtF_u \qquad \textbf{(10.3-2a)}$$

when deformation at the bolt hole at service load is not a design consideration:

$$R_n = 1.5L_c tF_u \leq 3.0\,dtF_u \qquad \textbf{(10.3-2b)}$$

(b) For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force:

$$R_n = 1.0L_c tF_u \leq 2.0\,dtF_u \qquad \textbf{(10.3-2c)}$$

In the foregoing,

R_n = nominal bearing strength of the connected material, kips (N)

F_u = specified minimum tensile strength of the connected material, MPa

L_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, mm

d = nominal bolt diameter, mm

t = thickness of connected material, mm

For connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts.

10.3.11 **Long Grips.** A307 bolts providing design strength, and for which the grip exceeds five diameters, shall have their number increased one percent for each additional 2 mm in the grip.

SECTION 10.4
DESIGN RUPTURE STRENGTH

10.4.1 **Shear Rupture Strength.** The design strength for the limit state of rupture along a shear failure path in the affected elements of connected members shall be taken as ϕR_n where

ϕ = 0.75

$R_n = F_u A_{nv}$

A_{nv} = net area subject to shear, mm^2

10.4.2 **Tension Rupture Strength.** The design strength for the limit state of rupture along a tension path in the affected elements of connected members shall be taken as ϕR_n where

ϕ = 0.75

$R_n = F_u A_{nt}$

A_{nt} = net area subject to tension, mm^2

10.4.3 **Block Shear Rupture Strength.** Block shear is a limit state in which the resistance is determined by the sum of the shear strength on a failure path(s) and the tensile strength on a perpendicular segment. It shall be checked at beam end connections where the top flange is coped and in similar situations, such as tension members and gusset plates. When ultimate rupture strength on the net section is used to determine the resistance on one segment, yielding on the gross section shall be used on the perpendicular segment. The block shear rupture design strength, ϕR_n, shall be determined as follows:

(a) When $F_u A_{nt} \geq 0.6 F_u A_{nv}$:

$$\phi R_n = \phi\ [0.6 F_y A_{gv} + F_u A_{nt}] \leq \phi\ [0.6 \mathrm{F_y A_{nv}} + \mathrm{F_u A_{nt}}] \qquad \textbf{(10.4-1)}$$

(b) When $F_u A_{nt} < 0.6 F_u A_{nv}$:

$$\phi R_n = \phi\ [0.6 F_u A_{nv} + F_y \mathrm{A_{gt}}] \leq \phi\ [0.6 \mathrm{F_y A_{nv}} + \mathrm{F_u A_{nt}}] \qquad \textbf{(10.4-2)}$$

where

ϕ = 0.75

A_{gv} = gross area subject to shear, mm^2

A_{gt} = gross area subject to tension, mm^2

A_{nv} = net area subject to shear, mm^2

A_{nt} = net area subject to tension, mm^2

SECTION 10.5
CONNECTING ELEMENTS

This section applies to the design of connecting elements, such as plates, gussets, angles, brackets, and the panel zones of beam-to-column connections.

10.5.1 **Eccentric Connections.** Intersecting axially stressed members shall have their gravity axis intersect at one point, if practicable; if not, provision shall be made for bending and shearing stresses due to the eccentricity. Also see Section 10.1.8.

10.5.2 **Design Strength of Connecting Elements in Tension.** The design strength, ϕR_n, of welded, bolted, and riveted connecting elements statically loaded in tension (e.g., splice and gusset plates) shall be the lower value obtained according to limit states of yielding, rupture of the connecting element, and block shear rupture.

(a) For tension yielding of the connecting element: ϕ = 0.90

$$R_n = A_g F_y \qquad \textbf{(10.5-1)}$$

(b) For tension rupture of the connecting element: $\phi = 0.75$

$$R_n = A_n F_u \qquad \textbf{(10.5-2)}$$

where A_n is the net area, not to exceed $0.85A_g$.

(c) For block shear rupture of connecting elements, see Section 10.4.3.

10.5.3 **Other Connecting Elements.** For all other connecting elements, the design strength, ϕR_n, shall be determined for the applicable limit state to ensure that the design strength is equal to or greater than the required strength, where R_n is the nominal strength appropriate to the geometry and type of loading on the connecting element. For shear yielding of the connecting element:

$$\phi = 0.90$$

$$R_n = 0.60\, A_g F_y \qquad \textbf{(10.5-3)}$$

If the connecting element is in compression an appropriate limit state analysis shall be made.

SECTION 10.6
FILLERS

In welded construction, any filler 6 mm or more in thickness shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate load, applied at the surface of the filler. The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler less than 6 mm thick shall have its edges made flush with the edges of the splice plate and the weld size shall be the sum of the size necessary to carry the splice plus the thickness of the filler plate.

When a bolt that carries load passes through fillers that are equal to or less than 6 mm thick, the design shear strength shall be used without reduction. When a bolt that carries load passes through fillers that are greater than 6 mm thick, one of the following requirements shall apply:

(1) For fillers that are equal to or less than 19 mm thick, the design shear strength of the bolts shall be multiplied by the factor $[1 - 0.0154(t - 6)]$, where t is the total thickness of the fillers up to 19 mm.

(2) The fillers shall be extended beyond the joint and the filler extension shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross-section of the connected element and the fillers;

(3) The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or

(4) The joint shall be designed as a slip-critical joint.

SECTION 10.7
SPLICES

Groove-welded splices in plate girders and beams shall develop the full strength of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice.

SECTION 10.8
BEARING STRENGTH

The strength of surfaces in bearing is ϕR_n where

ϕ = 0.75

R_n is defined below for the various types of bearing

(a) For milled surfaces, pins in reamed, drilled, or bored holes, and ends of fitted bearing stiffeners,

$$R_n = 1.8F_y A_{pb} \quad \textbf{(10.8-1)}$$

where

F_y = specified minimum yield stress, MPa

A_{pb} = projected bearing area, mm^2

(b) For expansion rollers and rockers,

If $d \le 635$ mm,

$$R_n = 1.2\,(F_y - 90)ld\,/\,20 \quad \textbf{(10.8-2)}$$

If $d > 635$ mm,

$$R_n = 6.0(F_y - 90)l\sqrt{d}\,/\,20 \quad \textbf{(10.8-3)}$$

where

d = diameter, mm

l = length of bearing, mm

SECTION 10.9
COLUMN BASES AND BEARING ON CONCRETE

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

In the absence of code regulations, design bearing loads on concrete may be taken as $\phi_c P_p$

(a) On the full area of a concrete support

$$P_p = 0.85 f_c' A_1 \quad \textbf{(10.9-1)}$$

(b) On less than the full area of a concrete support

$$P_p = 0.85 f_c' A_1 \sqrt{(A_2 / A_1)} \quad \textbf{(10.9-2)}$$

where

ϕ_c = 0.60

A_1 = area of steel concentrically bearing on a concrete support, mm^2

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm^2

$\sqrt{(A_2 / A_1)} \le 2$

SECTION 10.10
ANCHOR RODS AND EMBEDMENTS

Steel anchor rods and embedments shall be proportioned to develop the factored load combinations stipulated in Section 1.4. If the load factors and combinations stipulated in Section 1.4 are used to design concrete structural elements, the provisions of SBC-304 shall be used with appropriate ϕ factors as given in SBC-304.

CHAPTER 11
CONCENTRATED FORCES, PONDING, AND FATIGUE

This chapter covers member strength design considerations pertaining to concentrated forces, ponding, and fatigue.

SECTION 11.1
FLANGES AND WEBS WITH CONCENTRATED FORCES

11.1.1 **Design Basis.** Sections 11.1.2 through 11.1.7 apply to single and double concentrated forces as indicated in each Section. A single concentrated force is tensile or compressive. Double concentrated forces, one tensile and one compressive, form a couple on the same side of the loaded member.

Transverse stiffeners are required at locations of concentrated tensile forces in accordance with Section 11.1.2 for the limit state of flange local bending, and at unframed ends of beams and girders in accordance with Section 11.1.8. Transverse stiffeners or doubler plates are required at locations of concentrated forces in accordance with Sections 11.1.3 through 11.1.6 for the limit states of web local yielding, crippling, sidesway buckling, and compression buckling. Doubler plates or diagonal stiffeners are required in accordance with Section 11.1.7 for the limit state of web panel-zone shear.

Transverse stiffeners and diagonal stiffeners required by Sections 11.1.2 through 11.1.8 shall also meet the requirements of Section 11.1.9. Doubler plates required by Sections 11.1.3 through 11.1.6 shall also meet the requirements of Section 11.1.10.

11.1.2 **Flange Local Bending.** This Section applies to both tensile single-concentrated forces and the tensile component of double-concentrated forces.

A pair of transverse stiffeners extending at least one-half the depth of the web shall be provided adjacent to a concentrated tensile force centrally applied across the flange when the required strength of the flange exceeds ϕR_n,

where

ϕ = 0.90

$$R_n = 6.25\, t_f^2\, F_{yf} \qquad \textbf{(11.1-1)}$$

F_{yf} = specified minimum yield stress of the flange, MPa

t_f = thickness of the loaded flange, mm

If the length of loading across the member flange is less than $0.15b$, where b is the member flange width, Equation 11.1-1 need not be checked.

When the concentrated force to be resisted is applied at a distance from the member end that is less than $10t_f$, R_n shall be reduced by 50 percent.

When transverse stiffeners are required, they shall be welded to the loaded flange to develop the welded portion of the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, see Section 11.1.9.

11.1.3 **Web Local Yielding.** This Section applies to single-concentrated forces and both components of double-concentrated forces.

Either a pair of transverse stiffeners or a doubler plate, extending at least one-half the depth of the web, shall be provided adjacent to a concentrated tensile or compressive force when the required strength of the web at the toe of the fillet exceeds ϕR_n,

where

ϕ = 1.0

and R_n is determined as follows:

(a) When the concentrated force to be resisted is applied at a distance from the member end that is greater than the depth of the member d,

$$R_n = (5k + N)F_{yw}\ t_w \qquad \textbf{(11.1-2)}$$

(b) When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member d,

$$R_n = (2.5k + N)F_{yw}\ t_w \qquad \textbf{(11.1-3)}$$

In Equations 11.1-2 and 11.1-3, the following definitions apply:

F_{yw} = specified minimum yield stress of the web, MPa

N = length of bearing (not less than k for end beam reactions), mm

k = distance from outer face of the flange to the web toe of the fillet, mm

t_w = web thickness, mm

When required, for a tensile force normal to the flange, transverse stiffeners shall be welded to the loaded flange to develop the connected portion of the stiffener. When required for a compressive force normal to the flange, transverse stiffeners shall either bear on or be welded to the loaded flange to develop the force transmitted to the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, see Section 11.1.9.

Alternatively, when doubler plates are required, see Section 11.1.10.

11.1.4 **Web Crippling.** This Section applies to both compressive single-concentrated forces and the compressive component of double-concentrated forces.
Either a transverse stiffener, a pair of transverse stiffeners, or a doubler plate, extending at least one-half the depth of the web, shall be provided adjacent to a concentrated compressive force when the required strength of the web exceeds ϕR_n,

where

ϕ = 0.75

and R_n is determined as follows:

(a) When the concentrated compressive force to be resisted is applied at a distance from the member end that is greater than or equal to $d/2$,

$$R_n = 0.80 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} \qquad \textbf{(11.1-4)}$$

(b) When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than $d/2$,

For $N/d \leq 0.2$,

$$R_n = 0.40 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} \qquad \textbf{(11.1-5a)}$$

For $N/d > 0.2$,

$$R_n = 0.40 t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} \qquad \textbf{(11.1-5b)}$$

In Equations 11.1-4 and 11.1-5, the following definitions apply:

d = overall depth of the member, mm

t_f = flange thickness, mm

When transverse stiffeners are required, they shall either bear on or be welded to the loaded flange to develop the force transmitted to the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, see Section 11.1.9.

Alternatively, when doubler plates are required, see Section 11.1.10.

11.1.5 **Web Sidesway Buckling.** This Section applies only to compressive single-concentrated forces applied to members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated force.

The design strength of the web is ϕR_n,

where

ϕ = 0.85

and R_n is determined as follows:

(a) If the compression flange is restrained against rotation:

For $(h/t_w)/(l/b_f) \leq 2.3$,

$$R_n = \frac{C_r t_w^3 t_f}{h^2} \left[1 + 0.4\left(\frac{h/t_w}{l/b_f}\right)^3 \right] \qquad \textbf{(11.1-6)}$$

for $(h / t_w) / (l / b_f) > 2.3$, the limit state of sidesway web buckling does not apply. When the required strength of the web exceeds ϕR_n, local lateral bracing shall be provided at the tension flange or either a pair of transverse stiffeners or a doubler plate, extending at least one-half the depth of the web, shall be provided adjacent to the concentrated compressive force.

When transverse stiffeners are required, they shall either bear on or be welded to the loaded flange to develop the full-applied force. The weld connecting transverse stiffeners to the web shall be sized to transmit the force in the stiffener to the web. Also, see Section 11.1.9.

Alternatively, when doubler plates are required, they shall be sized to develop the full-applied force. Also, see Section 11.1.10.

(b) If the compression flange is *not* restrained against rotation:

For $(h / t_w) / (l / b_f) \leq 1.7$,

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right] \qquad \textbf{(11.1-7)}$$

for $(h / t_w) / (l / b_f) > 1.7$, the limit state of sidesway web buckling does not apply. When the required strength of the web exceeds ϕR_n local lateral bracing shall be provided at both flanges at the point of application of the concentrated forces.

In Equations 11.1-6 and 11.1-7, the following definitions apply:

l = largest laterally unbraced length along either flange at the point of load, mm

b_f = flange width, mm

t_f = flange thickness, mm

t_w = web thickness, mm

h = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of fasteners or the clear distance between flanges when welds are used for built-up shapes, mm

C_r = 6.62×10^6 MPa when $M_u < M_y$ at the location of the force

= 3.31×10^6 MPa when $M_u \geq M_y$ at the location of the force

11.1.6 **Web Compression Buckling.** This Section applies to a pair of compressive single-concentrated forces or the compressive components in a pair of double-concentrated forces, applied at both flanges of a member at the same location.

Either a single transverse stiffener, or pair of transverse stiffeners, or a doubler plate, extending the full depth of the web, shall be provided adjacent to concentrated compressive forces at both flanges when the required strength of the web exceed ϕR_n,

where

ϕ = 0.90

and

$$R_n = \frac{24t_w^3\sqrt{EF_{yw}}}{h} \qquad \textbf{(11.1-8)}$$

When the pair of concentrated compressive forces to be resisted is applied at a distance from the member end that is less than $d/2$, R_n shall be reduced by 50 percent. When transverse stiffeners are required, they shall either bear on or be welded to the loaded flange to develop the force transmitted to the stiffener. The weld connecting transverse stiffeners to the web shall be sized to transmit the unbalanced force in the stiffener to the web. Also, see Section 11.1.9.

Alternatively, when doubler plates are required, see Section 11.1.10.

11.1.7 **Web Panel-Zone Shear.** Either doubler plates or diagonal stiffeners shall be provided within the boundaries of the rigid connection of members whose webs lie in a common plane when the required strength exceeds ϕR_v,

where

ϕ = 0.90

and R_v is determined as follows:

(a) When the effect of panel-zone deformation on frame stability is *not* considered in the analysis,

For $P_u \le 0.4P_y$

$$R_v = 0.60F_y d_c t_w \qquad \textbf{(11.1-9)}$$

For $P_u > 0.4P_y$

$$R_v = 0.60F_y d_c t_w \left(1.4 - \frac{P_u}{P_y}\right) \qquad \textbf{(11.1-10)}$$

(b) When frame stability, including plastic panel-zone deformation, is considered in the analysis:

For $P_u \le 0.75P_y$

$$R_v = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^{\,2}}{d_b d_c t_w}\right) \qquad \textbf{(11.1-11)}$$

For $P_u > 0.75P_y$

$$R_v = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^{\,2}}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2P_u}{P_y}\right) \qquad \textbf{(11.1-12)}$$

In Equations 11.1-9 through 11.1-12, the following definitions apply:

t_w = column web thickness, in. (mm)

b_{cf} = width of column flange, in. (mm)

t_{cf} = thickness of the column flange, in. (mm)

d_b = beam depth, in. (mm)

d_c = column depth, in. (mm)

F_y = yield strength of the column web, ksi (MPa)

P_y = $F_y A$, axial yield strength of the column, kips (N)

A = column cross-sectional area, in.^{2} (mm^2)

When doubler plates are required, they shall meet the criteria of Section 6.2 and shall be welded to develop the proportion of the total shear force which is to be carried.

Alternatively, when diagonal stiffeners are required, the weld connecting diagonal stiffeners to the web shall be sized to transmit the stiffener force caused by unbalanced moments to the web. Also, see Section 11.1.9.

11.1.8 **Unframed Ends of Beams and Girders.** At unframed ends of beams and girders not otherwise restrained against rotation about their longitudinal axes, a pair of transverse stiffeners, extending the full depth of the web, shall be provided. Also, see Section 11.1.9.

11.1.9 **Additional Stiffener Requirements for Concentrated Forces.** Transverse and diagonal stiffeners shall also comply with the following criteria:

(1) The width of each stiffener plus one-half the thickness of the column web shall not be less than one-third of the width of the flange or moment connection plate delivering the concentrated force.

(2) The thickness of a stiffener shall not be less than one-half the thickness of the flange or moment connection plate delivering the concentrated load, and not less than its width times $1.79\sqrt{F_y / E}$.

Full depth transverse stiffeners for compressive forces applied to a beam or plate girder flange shall be designed as axially compressed members (columns) in accordance with the requirements of Section 5.2 with an effective length of $0.75h$, a cross section composed of two stiffeners and a strip of the web having a width of $25t_w$ at interior stiffeners and $12t_w$ at the ends of members.

The weld connecting bearing stiffeners to the web shall be sized to transmit the excess web shear force to the stiffener. For fitted bearing stiffeners, see Section 10.8.

11.1.10 **Additional Doubler Plate Requirements for Concentrated Forces.** Doubler plates required by Sections 11.1.3 through 11.1.6 shall also comply with the following criteria:

(1) The thickness and extent of the doubler plate shall provide the additional material necessary to equal or exceed the strength requirements.

(2) The doubler plate shall be welded to develop the proportion of the total force transmitted to the doubler plate.

SECTION 11.2
PONDING

The roof system shall be investigated by structural analysis to assure adequate strength and stability under ponding conditions, unless the roof surface is provided with sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rainwater.

The roof system shall be considered stable and no further investigation is needed if:

$$C_p + 0.9C_s \leq 0.25 \quad \textbf{(11.2-1)}$$

$$I_d \geq 3\,940\,S^4 \quad \textbf{(11.2-2)}$$

where

C_p = $\dfrac{504 L_s L_p^4}{I_p}$

C_s = $\dfrac{504 S L_s^4}{I_s}$

L_p = column spacing in direction of girder (length of primary members), m

L_S = column spacing perpendicular to direction of girder (length of secondary members), m

S = spacing of secondary members, m

I_p = moment of inertia of primary members, mm^4

I_S = moment of inertia of secondary members, mm^4

I_d = moment of inertia of the steel deck supported on secondary members, mm^4 per m

For trusses and steel joists, the moment of inertia I_S shall be decreased 15 percent when used in the above equation. A steel deck shall be considered a secondary member when it is directly supported by the primary members.

SECTION 11.3
DESIGN FOR CYCLIC LOADING (FATIGUE)

Few members or connections in conventional buildings need to be designed for fatigue, since most load changes in such structures occur only a small number of times or produce only minor stress fluctuations. The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design. However, crane runways and supporting structures for machinery and equipment are often subject to fatigue loading conditions.

This section applies to members and connections subject to high cycle loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure (fatigue).

11.3.1 **General.** The provisions of this section apply to stresses calculated on the basis of unfactored loads. The maximum permitted stress due to unfactored loads is $0.66F_y$.

Stress range is defined as the magnitude of the change in stress due to the application or removal of the unfactored live load. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the numerical sum of maximum shearing stresses of opposite direction at the point of probable crack initiation.

In the case of complete-joint-penetration butt welds, the maximum design stress range calculated by Equation 11.3-1 applies only to welds with internal soundness meeting the acceptance requirements of Section 6.12.2 or 6.13.2 of AWS D1.1. No evaluation of fatigue resistance is required if the live load stress range is less than

the threshold stress range, F_{TH}. See Table 11.3-1.

No evaluation of fatigue resistance is required if the number of cycles of application of live load is less than 2 x 10^4.

The cyclic load resistance determined by this provision is applicable to structures with suitable corrosion protection or subject only to mildly corrosive atmospheres, such as normal atmospheric conditions.

The cyclic load resistance determined by this provision is applicable only to structures subject to temperatures not exceeding 150°C.

The Engineer of Record shall provide either complete details including weld sizes or shall specify the planned cycle life and the maximum range of moments, shears and reactions for the connections.

11.3.2 **Calculation of Maximum Stresses and Stress Ranges.** Calculated stresses shall be based upon elastic analysis. Stresses shall not be amplified by stress concentration factors for geometrical discontinuities.

For bolts and threaded rods subject to axial tension, the calculated stresses shall include the effects of prying action, if any.

In the case of axial stress combined with bending, the maximum stresses, of each kind, shall be those determined for concurrent arrangements of the applied load. For members having symmetric cross sections, the fasteners and welds shall be arranged symmetrically about the axis of the member, or the total stresses including those due to eccentricity shall be included in the calculation of the stress range.

For axially loaded angle members where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to joint eccentricity, shall be included in the calculation of stress range.

11.3.3 **Design Stress Range:** The range of stress at service loads shall not exceed the stress range computed as follows.

(a) For stress categories A, B, B´, C, D, E and E´ the design stress range, F_{SR}, shall be determined by Equation 11.3-1.

$$F_{SR} = \left(\frac{C_f \times 327}{N} \right)^{0.333} \geq F_{TH} \qquad \textbf{(11.3-1)}$$

where

F_{SR} = Design stress range, MPa

C_f = Constant from Table 11.3-1 for the category

N = Number of stress range fluctuations in design life

= Number of stress range fluctuations per day × 365 × years of design life

F_{TH} = Threshold fatigue stress range, maximum stress range for indefinite design life from Table 11.3-1, MPa

(b) For stress Category F, the design stress range, F_{SR}, shall be determined by Equation 11.3-2.

$$F_{SR} = \left(\frac{C_f \times 11 \times 10^4}{N} \right)^{0.167} \geq F_{TH} \tag{11.3-2}$$

(c) For tension-loaded plate elements connected at their end by cruciform, T- or corner details with complete-joint-penetration groove welds or partial-joint-penetration groove welds, fillet welds, or combinations of the preceding, transverse to the direction of stress, the design stress range on the cross section of the tension-loaded plate element at the toe of the weld shall be determined as follows:

Based upon crack initiation from the toe of the weld on the tension loaded plate element the design stress range, F_{SR}, shall be determined by Equation 11.3-1, for Category C which is equal to

$$F_{SR} = \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \geq 68.9$$

Based upon crack initiation from the root of the weld the design stress range, F_{SR}, on the tension loaded plate element using transverse partial-joint-penetration groove welds, with or without reinforcing or contouring fillet welds, the design stress range on the cross section at the toe of the weld shall be determined by Equation 11.3-3, Category C$'$ as follows:

$$F_{SR} = 1.72 R_{PJP} \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \tag{11.3-3}$$

where

R_{PJP} = reduction factor for reinforced or non-reinforced transverse partial-joint-penetration (PJP) joints. Use Category C if R_{PJP} = 1.0.

$$= \left(\frac{0.65 - 0.59\left(\frac{2a}{t_p}\right) + 0.72\left(\frac{w}{t_p}\right)}{t_p^{0.167}} \right) \leq 1.0$$

$2a$ = the length of the non-welded root face in the direction of the thickness of the tension-loaded plate, mm

w = the leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, mm

t_p = thickness of tension loaded plate, mm

Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element the design stress range, F_{SR}, on the cross section at the toe of the welds shall be determined by Equation 11.3-4, Category C as follows:

$$F_{SR} = 1.72 R_{FIL} \left(\frac{14.4 \times 10^{11}}{N} \right)^{0.333} \tag{11.3-4}$$

where

R_{FIL} = reduction factor for joints using a pair of transverse fillet welds only. Use Category C if R_{FIL} = 1.0.

$$R_{FIL} = \left(\frac{0.06 + 0.72(w/t_p)}{t_p^{0.167}} \right) \leq 1.0$$

11.3.4 **Bolts and Threaded Parts.** The range of stress at service loads shall not exceed the stress range computed as follows.

(a) For mechanically fastened connections loaded in shear, the maximum range of stress in the connected material at service loads shall not exceed the design stress range computed using Equation 11.3-1 where C_f and F_{TH} are taken from Section 2 of Table 11.3-1.

(b) For high-strength bolts, common bolts, and threaded anchor rods with cut, ground or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial load and moment plus load due to prying action shall not exceed the design stress range computed using Equation 11.3-1. The factor C_f shall be taken as $3.9x10^8$ (as for Category E). The threshold stress, F_{TH} shall be taken as 48 MPa (as for category D). The net tensile area is given by Equation 11.3-5.

$$A_t = \frac{\pi}{4} (d_b - 0.9382\text{P})^2 \qquad \textbf{(11.3-5)}$$

where

P = pitch, mm per thread

d_b = the nominal diameter (body or shank diameter),mm

For joints in which the material within the grip is not limited to steel or joints which are not tensioned to the requirements of Table 10.3-1, all axial load and moment applied to the joint plus effects of prying action (if any) shall be assumed to be carried exclusively by the bolts or rods.

For joints in which the material within the grip is limited to steel and which are tensioned to the requirements of Table 10.3-1, an analysis of the relative stiffness of the connected parts and bolts shall be permitted to be used to determine the tensile stress range in the pretensioned bolts due to the total service live load and moment plus effects of prying action. Alternatively, the stress range in the bolts shall be assumed to be equal to the stress on the net tensile area due to 20 percent of the absolute value of the service load axial load and moment from dead, live and other loads.

11.3.5 **Special Fabrication and Erection Requirements.** Longitudinal backing bars are permitted to remain in place, and if used, shall be continuous. If splicing is necessary for long joints, the bar shall be joined with complete penetration butt joints and the reinforcement ground prior to assembly in the joint.

In transverse joints subject to tension, backing bars, if used, shall be removed and the joint back gouged and welded.

In transverse complete-joint-penetration T- and corner joints, a reinforcing fillet weld, not less than 6 mm in size shall be added at re-entrant corners.

The surface roughness of flame cut edges subject to significant cyclic tensile stress ranges shall not exceed 25 μm, where ASME B46.1 is the reference standard.

Re-entrant corners at cuts, copes and weld access holes shall form a radius of not less than 10 mm by pre-drilling or sub-punching and reaming a hole, or by thermal

cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal surface.

For transverse butt joints in regions of high tensile stress, run-off tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Run-off tabs shall be removed and the end of the weld finished flush with the edge of the member.

See Section 10.2.2.2 *Fillet Weld Terminations* for requirements for end returns on certain fillet welds subject to cyclic service loading.

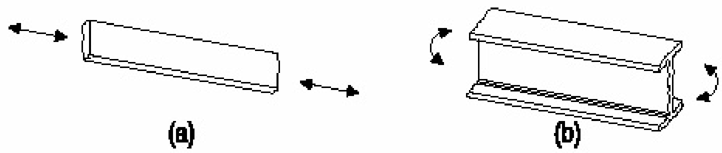
Table 11.3-1
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING				
1.1 Base metal, except non-coated weathering steel, with rolled or cleaned surface. Flame-cut edges with surface roughness value of ($25 \mu m$) or less, but without re-entrant corners.	A	250×10^8	165	Away from all welds or structural connections
1.2 Non-coated weathering steel base metal with rolled or cleaned surface. Flame-cut edges with surface roughness value of $1000 \mu in$ ($25 \mu m$) or less, but without re-entrant corners.	B	120×10^8	110	Away from all welds or structural connections
1.3 Member with drilled or reamed holes. Member with re-entrant corners at copes, cuts, block-outs or other geometrical discontinuities made to requirements of Section 11.3.5, except weld access holes.	B	120×10^8	110	At any external edge or at hole perimeter
1.4 Rolled cross sections with weld access holes made to requirements of Sections 10.1.6 and 11.3.5. Members with drilled or reamed holes containing bolts for attachment of light bracing where there is a small longitudinal component of brace force.	C	44×10^8	69	At re-entrant corner of weld access hole or at any small hole (may contain bolt for minor connections)
SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS				
2.1 Gross area of base metal in lap joints connected by high-strength bolts in joints satisfying all requirements for slip-critical connections.	B	120×10^8	110	Through gross section near hole
2.2 Base metal at net section of high-strength bolted joints, designed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections.	B	120×10^8	110	In net section originating at side of hole
2.3 Base metal at the net section of other mechanically fastened joints except eye bars and pin plates.	D	22×10^8	48	In net section originating at side of hole
2.4 Base metal at net section of eyebar head or pin plate.	E	11×10^8	31	In net section originating at side of hole

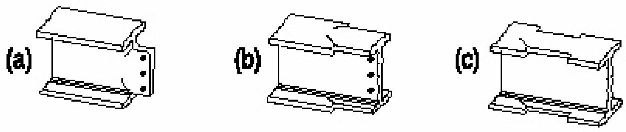
Table 11.3-1(Cont'd)
Fatigue Design Parameters
Illustrative Typical Examples

SECTION 1 - PLAIN MATERIAL AWAY FROM ANY WELDING

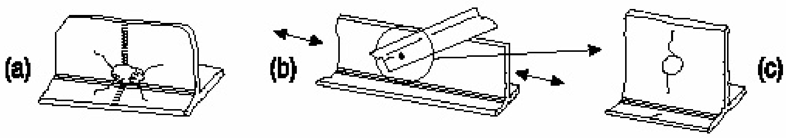
1.1 and 1.2



1.3

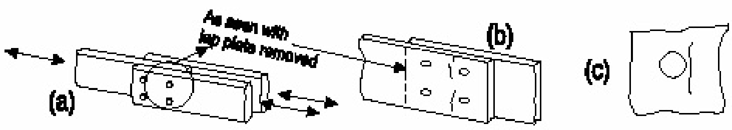


1.4

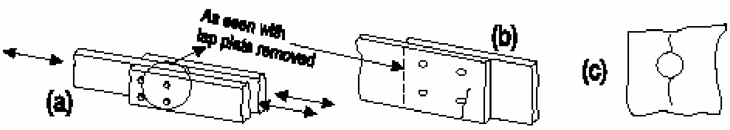


SECTION 2 - CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS

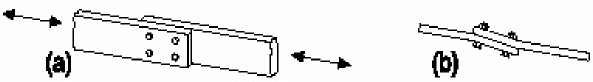
2.1



2.2



2.3



2.4

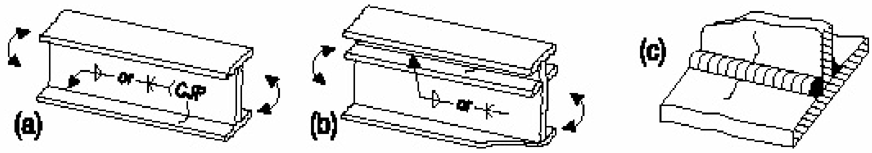
Table 11.3-1(Cont'd)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS				
3.1 Base metal and weld metal in members without attachments built-up of plates or shapes connected by continuous longitudinal complete-joint-penetration groove welds, back gouged and welded from second side, or by continuous fillet welds.	B	120×10^8	110	From surface or internal discontinuities in weld away from end of weld
3.2 Base metal and weld metal in members without attachments built-up of plates or shapes, connected by continuous longitudinal complete penetration groove welds with backing bars not removed, or by continuous partial-joint-penetration groove welds.	B′	61×10^8	83	From surface or internal discontinuities in weld, including weld attaching backing bars
3.3 Base metal and weld metal termination of longitudinal welds at weld access holes in connected built-up members.	D	22×10^8	48	From the weld termination into the web or flange
3.4 Base metal at ends of longitudinal intermittent fillet weld segments.	E	11×10^8	31	In connected material at start and stop locations of any weld deposit
3.5 Base metal at ends of partial length welded cover-plates narrower than the flange having square or tapered ends, with or without welds across the ends of cover-plates wider than the flange with welds across the ends.				In flange at toe of end weld or in flange at termination of longitudinal weld or in edge of flange with wide cover-plates
Flange thickness $\leq$ 20 mm	E	11×10^8	31	
Flange thickness > 20 mm	E′	3.9×10^8	18	
3.6 Base metal at ends of partial length welded cover-plates wider than the flange without welds across the ends.	E′	3.9×10^8	18	In edge of flange at end of cover-plate weld
SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections. Welds shall be on each side of the axis of the member to balance weld stresses.				Initiating from end of any weld termination extending into the base metal
$t \leq$ 13 mm	E	11×10^8	31	
t > 13 mm	E′	3.9×10^8	18	

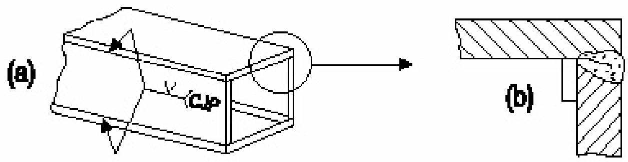
Table 11.3-1(Cont'd)
Fatigue Design Parameters
Illustrative Typical Examples

SECTION 3 - WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS

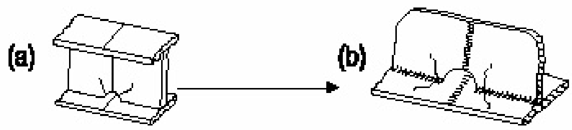
3.1



3.2



3.3



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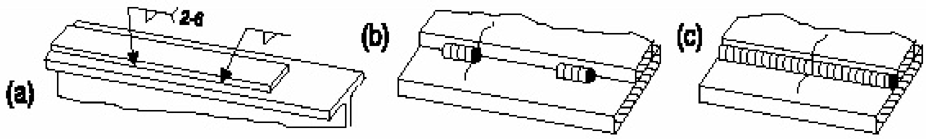
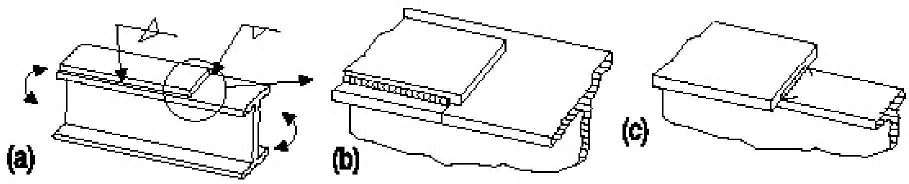
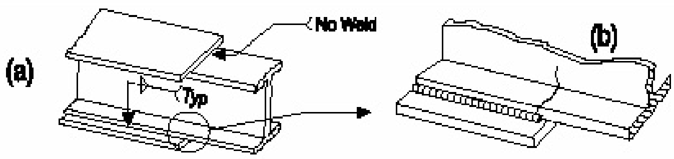


3.5



3.6



SECTION 4 - LONGITUDINAL FILLET WELDED END CONNECTIONS

4.1

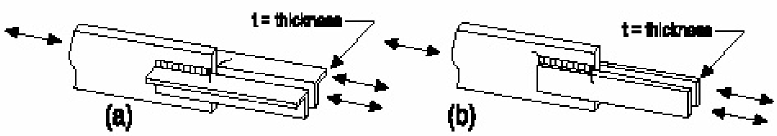


Table 11.3-1(Cont'd)
Fatigue Design Parameters

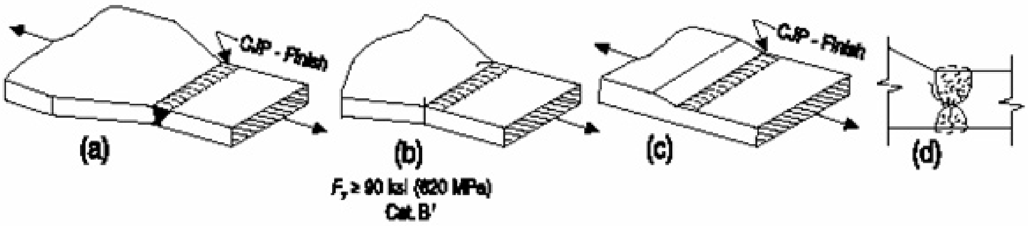
Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.1 Base metal and weld metal in or adjacent to complete joint penetration groove welded splices in rolled or welded cross sections with welds ground essentially parallel to the direction of stress.	B	120×10^{a}	110	From internal discontinuities in filler metal or along the fusion boundary
5.2 Base metal and weld metal in or adjacent to complete joint penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in thickness or width made on a slope no greater than 8 to 20%.				From internal discontinuities in filler metal or along the fusion boundary or at start of transition when $F_y \geq$ 620 MPa
F_y < 620 MPa	B	120×10^{8}	110	
$F_y \geq$ 620 MPa	B'	61×10^{8}	83	
5.3 Base metal with F_y equal to or greater than 90 ksi (620 MPa) and weld metal in or adjacent to complete joint penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in width made on a radius of not less than 2 ft. (600 mm) with the point of tangency at the end of the groove weld.	B	120×10^{a}	110	From internal discontinuities in filler metal or discontinuities along the fusion boundary
5.4 Base metal and weld metal in or adjacent to the toe of complete joint penetration T or corner joints or splices, with or without transitions in thickness having slopes no greater than 8 to 20%, when weld reinforcement is not removed.	C	44×10^{8}	69	From surface discontinuity at toe of weld extending into base metal or along fusion boundary
5.5 Base metal and weld metal at transverse end connections of tension-loaded plate elements using partial joint penetration butt or T or corner joints, with reinforcing or contouring fillets, F_{SR} shall be the smaller of the toe crack or root crack stress range.				Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at weld root subject to tension extending up and then out through weld
Crack initiating from weld toe:	C	44×10^{8}	69	
Crack initiating from weld root:	C′	(Equation 11.3-3)	None provided	

Table 11.3-1(Cont'd)
Fatigue Design Parameters
Illustrative Typical Examples

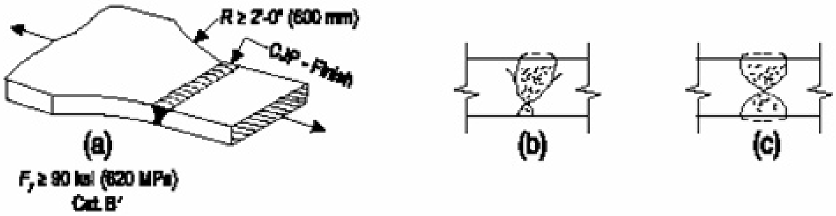
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS

5.1

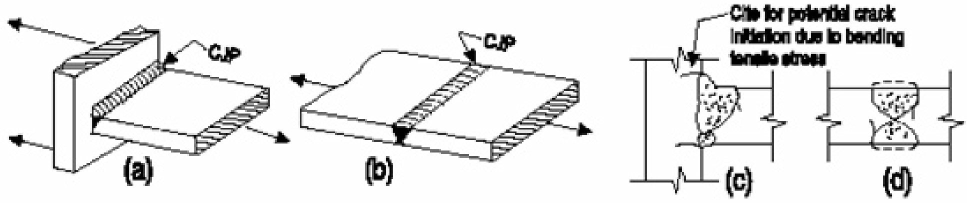
5.2



5.3



5.4



5.5

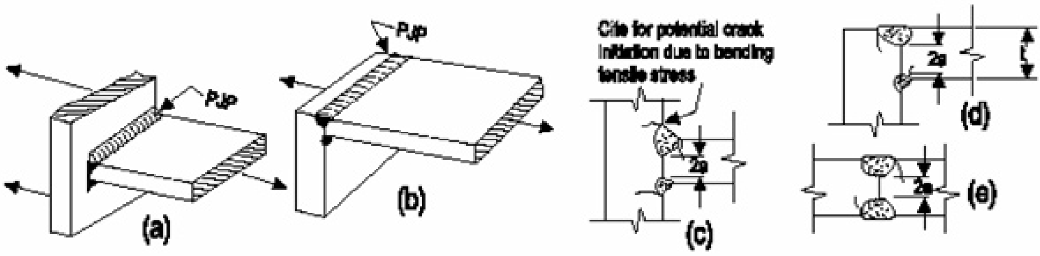


Table 11.3-1(Cont'd)
Fatigue Design Parameters

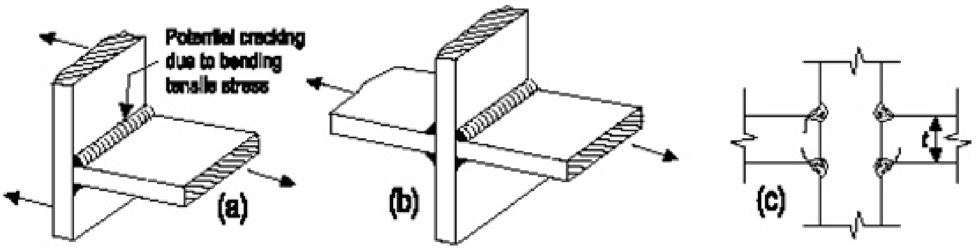
Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.6 Base metal and filler metal at transverse end connections of tension-loaded plate elements using a pair of fillet welds on opposite sides of the plate. F_{SR} shall be the smaller of the toe crack or root crack stress range.				Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at weld root subject to tension extending up and then out through weld
Crack initiating from weld toe:	C	44 x 10^8	69	
Crack initiating from weld root:	C''	(Equation 11.3-4)	None provided	
5.7 Base metal of tension loaded plate elements and on girders and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners.	C	44 x 10^8	69	From geometrical discontinuity at toe of fillet extending into base metal
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.1 Base metal at details attached by complete joint penetration groove welds subject to longitudinal loading only when the detail embodies a transition radius R with the weld termination ground smooth.				Near point of tangency of radius at edge of member
$R \geq 600$ mm	B	120 x 10^8	110	
600 mm > $R \geq 150$ mm	C	44 x 10^8	69	
150 mm > $R \geq 50$ mm	D	22 x 10^8	48	
50 mm > R	E	11 x 10^8	31	

Table 11.3-1(Cont'd)
Fatigue Design Parameters

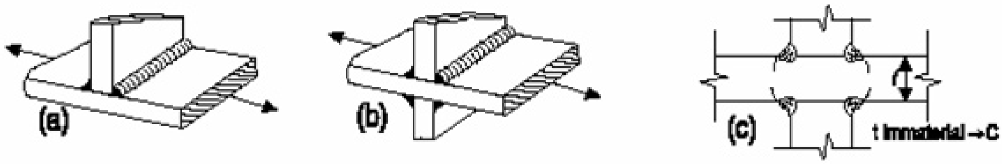
Illustrative Typical Examples

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)

5.6



5.7



SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS

6.1

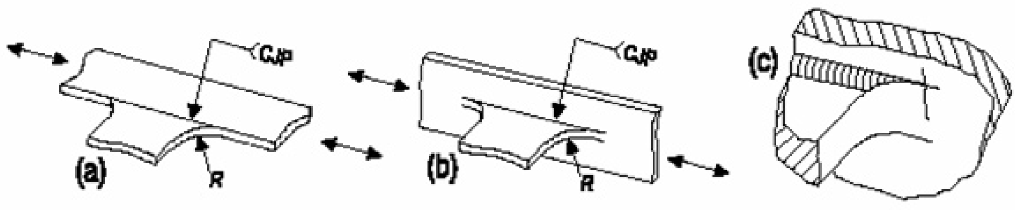


Table 11.3-1(Cont'd)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.2 Base metal at details of equal thickness attached by complete joint penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius *R* with the weld termination ground smooth.				
When weld reinforcement is removed:				
$R \geq 600$ mm	B	120×10^8	110	Near points of tangency of radius or in the weld or at fusion boundary or member or attachment
$600 \text{ mm} > R \geq 150$ mm	C	44×10^8	69	
$150 \text{ mm} > R \geq 50$ mm	D	22×10^8	48	
$50 \text{ mm} > R$	E	11×10^8	31	
When weld reinforcement is not removed:				
$R \geq 600$ mm	C	44×10^8	69	At toe of the weld either along edge of member or the attachment
$600 \text{ mm} > R \geq 150$ mm	C	44×10^8	69	
$150 \text{ mm} > R \geq 50$ mm	D	22×10^8	48	
$50 \text{ mm} > R$	E	11×10^8	31	
6.3 Base metal at details of unequal thickness attached by complete joint penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius R with the weld termination ground smooth.				
When weld reinforcement is removed:				
$R \geq 50$ mm	D	22×10^8	48	At toe of weld along edge of thinner material
$R \geq 50$ mm	E	11×10^8	31	In weld termination in small radius
When reinforcement is not removed:				
$R \geq 600$ mm	E	11×10^8	31	At toe of weld along edge of thinner material

Table 11.3-1(Cont'd)
Fatigue Design Parameters
Illustrative Typical Examples

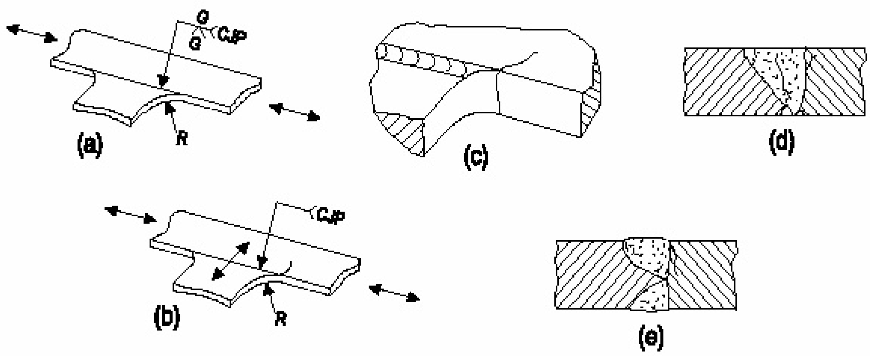
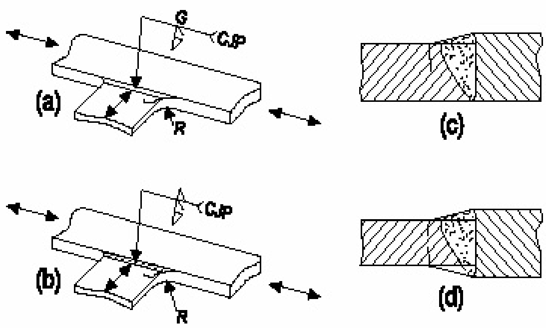
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)
6.2 
6.3 

Table 11.3-1(Cont'd)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} MPa	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.4 Base metal subject to longitudinal stress at transverse stress, attached by fillet or partial penetration groove welds parallel to direction of stress when the detail embodies a transition radius *R*, with weld termination ground smooth.				In weld termination or from the toe of the weld extending into member
$R > 50$ mm	D	22×10^8	48	
$R \leq 50$ mm	E	11×10^8	31	
SECTION 7 – BASE METAL AT SHORT ATTACHMENTS[1]				
7.1 Base metal subject to longitudinal loading at details attached by complete penetration groove welds parallel to direction of stress where the detail embodies a transition radius, *R*, less than 50 mm, and with detail length in direction of stress, a, and attachment height normal to surface of member, *b*:				In the member at the end of the weld
a < 50 mm	C	44×10^8	69	
50 mm $\leq$ a $\leq$ 12*b* or 100 mm	D	22×10^8	48	
a > 12*b* or 100 mm when *b* is $\leq$ 25 mm	E	11×10^8	31	
a > 12*b* or 100 mm when *b* is > 25 mm	E	3.9×10^8	18	
7.2 Base metal subject to longitudinal stress at details attached by fillet or partial joint penetration groove welds, with or without transverse load on detail, when the detail embodies a transition radius, *R*, with weld termination ground smooth:				In weld termination extending into member
$R > 50$ mm	D	22×10^8	48	
$R \leq 50$ mm	E	11×10^8	31	

[1] "Attachment" as used herein, is defined as any steel detail welded to a member which, by its mere presence and independent of its loading, causes a discontinuity in the stress flow in the member and thus reduces the fatigue resistance.

Table 11.3-1(Cont'd)
Fatigue Design Parameters
Illustrative Typical Examples

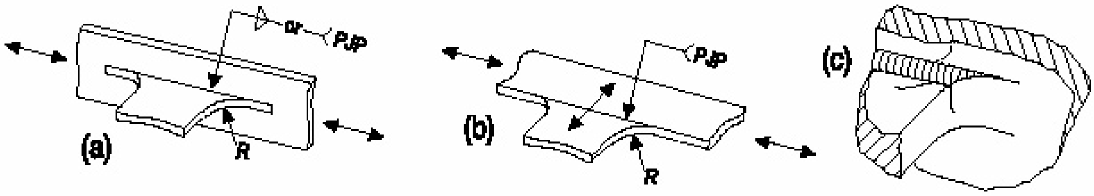
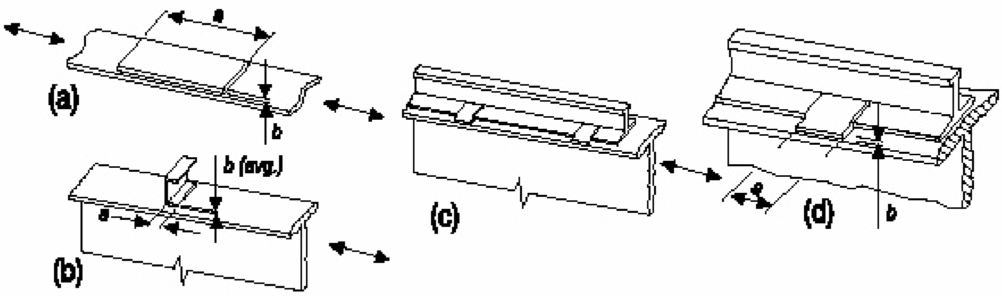
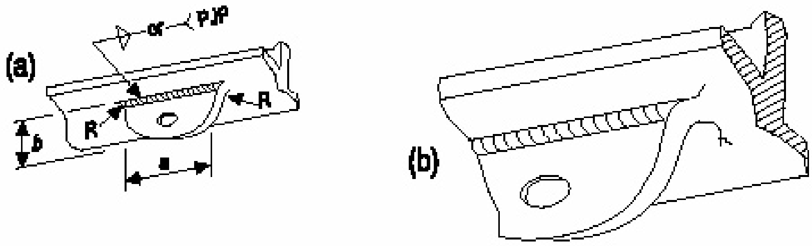
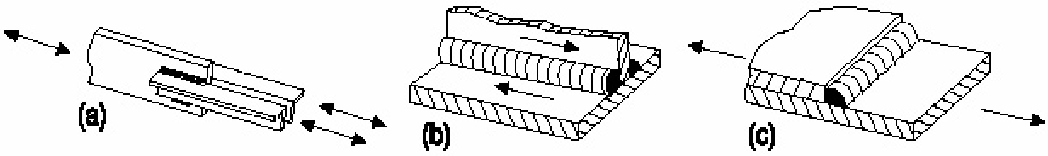
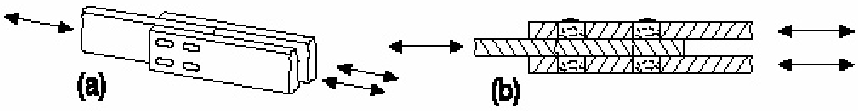
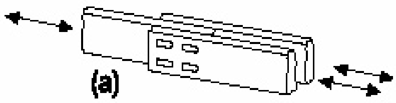
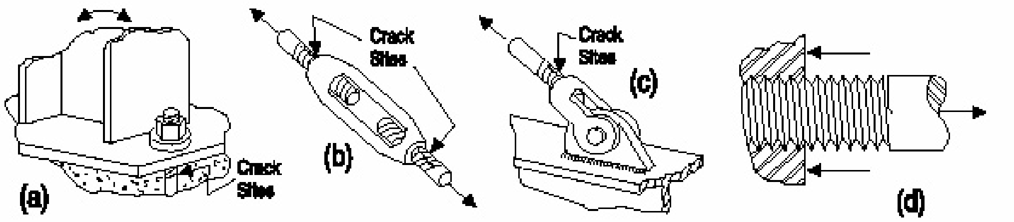
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)
6.4 
SECTION 7 – BASE METAL AT SHORT ATTACHMENTS[1]
7.1 
7.2 

Table 11.3-1(Cont'd) Fatigue Design Parameters				
Description	**Stress Category**	**Constant C_f**	**Threshold F_{TH} MPa**	**Potential Crack Initiation Point**
SECTION 8 – MISCELLANEOUS				
8.1 Base metal at stud-type shear connectors attached by fillet or electric stud welding	C	44×10^a	69	At toe of weld in base metal
8.2 Shear on throat of continuous or intermittent longitudinal or transverse fillet welds.	F	150×10^8 (Equation 11.3-2)	55	In throat of weld
8.3 Base metal at plug or slot welds.	E	11×10^8	31	At end of weld in base metal
8.4 Shear on plug or slot welds.	F	150×10^8 (Equation 11.3-2)	55	At faying surface
8.5 Not fully-tightened high-strength bolts, common bolts, threaded anchor rods and hanger rods with cut, ground or rolled threads. Stress range on tensile stress area due to live load plus prying action when applicable.	E'	3.9×10^8	48	At the root of the threads extending into the tensile stress area

Table 11.3-1(Cont'd) Fatigue Design Parameters Illustrative Typical Examples	
SECTION 8 - MISCELLANEOUS	
8.1	
8.2	
8.3	
8.4	
8.5	

CHAPTER 12
SERVICEABILITY DESIGN CONSIDERATIONS

This chapter is intended to provide design guidance for serviceability considerations.

Serviceability is a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage.

Limiting values of structural behavior to ensure serviceability (e.g., maximum deflections, accelerations, etc.) shall be chosen with due regard to the intended function of the structure. Where necessary, serviceability shall be checked using realistic loads for the appropriate serviceability limit state.

It is difficult to specify limiting values of structural performance based on serviceability considerations because these depend to a great extent on the type of structure, its intended use, and subjective physiological reaction. For example, acceptable structural motion in a hospital clearly would be much less than in an ordinary industrial building. It should be noted that humans perceive levels of structural motion that are far less than motions that would cause any structural damage. Serviceability limits must be determined through careful consideration by the designer and client.

SECTION 12.1
CAMBER

If any special camber requirements are necessary to bring a loaded member into proper relation with the work of other trades, as for the attachment of runs of sash, the requirements shall be set forth in the design documents. If camber involves the erection of any member under a preload, this shall be noted in the design documents.

Beams and trusses detailed without specified camber shall be fabricated so that after erection any camber due to rolling or shop assembly shall be upward.

SECTION 12.2
EXPANSION AND CONTRACTION

Adequate provision shall be made for expansion and contraction appropriate to the service conditions of the structure.

SECTION 12.3
DEFLECTIONS, VIBRATION, AND DRIFT

12.3.1 **Deflections.** Deformations in structural members and structural systems due to service loads shall not impair the serviceability of the structure.

12.3.2 **Floor Vibration.** Vibration shall be considered in designing beams and girders supporting large areas free of partitions or other sources of damping where excessive vibration due to pedestrian traffic or other sources within the building is not acceptable.

12.3.3 **Drift.** Lateral deflection or drift of structures due to code-specified wind or seismic loads shall not cause collision with adjacent structures nor exceed the limiting values of such drifts which may be specified or appropriate.

SECTION 12.4
CONNECTION SLIP

For the design of slip-critical connections see Sections 10.3.8 and 10.3.9.

SECTION 12.5
CORROSION

When appropriate, structural components shall be designed to tolerate corrosion or shall be protected against corrosion that may impair the strength or serviceability of the structure.

CHAPTER 13
FABRICATION, ERECTION, AND QUALITY CONTROL

This chapter provides requirements for shop drawings, fabrication, shop painting, erection, and quality control.

SECTION 13.1
SHOP DRAWINGS

Shop drawings giving complete information necessary for the fabrication of the component parts of the structure, including the location, type, and size of all welds, bolts, and rivets, shall be prepared in advance of the actual fabrication. These drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify pretensioned and slip-critical high-strength bolted connections. Shop drawings shall be made in conformity with good practice and with due regard to speed and economy in fabrication and erection.

SECTION 13.2
FABRICATION

13.2.1 **Cambering, Curving, and Straightening.** Local application of heat or mechanical means is permitted to be used to introduce or correct camber, curvature, and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 593°C for ASTM A514/A514M and ASTM A852/A852M steel nor 649°C for other steels.

13.2.2 **Thermal Cutting.** Thermally cut edges shall meet the requirements of AWS 5.15.1.2, 5.15.4.3 and 5.15.4.4 with the exception that thermally cut free edges which will be subject to calculated static tensile stress shall be free of round bottom gouges greater than 5 mm deep and sharp V-shaped notches. Gouges greater than 5 mm deep and notches shall be removed by grinding or repaired by welding.

Re-entrant corners, except re-entrant corners of beam copes and weld access holes, shall meet the requirements of AWS 5.16. If another specified contour is required it must be shown on the contract documents.

Beam copes and weld access holes shall meet the geometrical requirements of Section 10.1.6. For beam copes and weld access holes in ASTM A6/A6M Group 4 and 5 shapes and welded built-up shapes with material thickness greater than 50 mm, a preheat temperature of not less than 66°C shall be applied prior to thermal cutting.

13.2.3 **Planning of Edges.** Planning or finishing of sheared or thermally cut edges of plates or shapes is not required unless specifically called for in the design documents or included in a stipulated edge preparation for welding.

13.2.4 **Welded Construction.** The technique of welding, the workmanship, appearance, and quality of welds and the methods used in correcting nonconforming work shall be in accordance with AWS D1.1 except as modified in Section 10.2.

13.2.5 **Bolted Construction.** All parts of bolted members shall be pinned or bolted and rigidly held together during assembly. Use of a drift pin in bolt holes during

assembly shall not distort the metal or enlarge the holes. Poor matching of holes shall be cause for rejection.

If the thickness of the material is not greater than the nominal diameter of the bolt plus 3 mm, the holes are permitted to be punched. If the thickness of the material is greater than the nominal diameter of the bolt plus 3 mm, the holes shall be either drilled or sub-punched and reamed. The die for all sub-punched holes, and the drill for all sub-drilled holes, shall be at least 2 mm smaller than the nominal diameter of the bolt. Holes in ASTM A514/A514M steel plates over 13 mm thick shall be drilled.

Fully-inserted finger shims, with a total thickness of not more than 6 mm within a joint, are permitted in joints without changing the design strength (based upon hole type) for the design of connections. The orientation of such shims is independent of the direction of application of the load.

The use of high-strength bolts shall conform to the requirements of the RCSC *Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts*.

13.2.6 **Compression Joints.** Compression joints which depend on contact bearing as part of the splice strength shall have the bearing surfaces of individual fabricated pieces prepared by milling, sawing, or other suitable means except where the surface is mill finished faces of plates or sections which are free from surface unevenness and mill scales.

13.2.7 **Dimensional Tolerances.** Dimensional tolerances shall be in accordance with the AISC *Code of Standard Practice*.

13.2.8 **Finish of Column Bases.** Column bases and base plates shall be finished in accordance with the following requirements:

(1) Steel bearing plates 50 mm or less in thickness are permitted without milling, provided a satisfactory contact bearing is obtained. Steel bearing plates over 50 mm but not over 100 mm in thickness are permitted to be straightened by pressing or, if presses are not available, by milling for all bearing surfaces (except as noted in subparagraphs 2 and 3 of this section), to obtain a satisfactory contact bearing. Steel bearing plates over 100 mm in thickness shall be milled for all bearing surfaces (except as noted in subparagraphs 2 and 3 of this section).

(2) Bottom surfaces of bearing plates and column bases which are grouted to ensure full bearing contact on foundations need not be milled.

(3) Top surfaces of bearing plates need not be milled when complete-joint-penetration groove welds are provided between the column and the bearing plate.

SECTION 13.3
SHOP PAINTING

13.3.1 **General Requirements.** Shop painting and surface preparation shall be in accordance with the provisions of the AISC *Code of Standard Practice*.
Shop paint is not required unless specified by the contract documents.

13.3.2 **Inaccessible Surfaces.** Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the design documents.

13.3.3 **Contact Surfaces.** Paint is permitted unconditionally in bearing-type connections. For slip-critical connections, the faying surface requirements shall be in accordance with the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, paragraph 3(b).

13.3.4 **Finished Surfaces.** Machine-finished surfaces shall be protected against corrosion by a rust inhibitive coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary.

13.3.5 **Surfaces Adjacent to Field Welds.** Unless otherwise specified in the design documents, surfaces within 50 mm of any field weld location shall be free of materials that would prevent proper welding or produce objectionable fumes during welding.

SECTION 13.4
ERECTION

13.4.1 **Alignment of Column Bases.** Column bases shall be set level and to correct elevation with full bearing on concrete or masonry.

13.4.2 **Bracing.** The frame of steel skeleton buildings shall be carried up true and plumb within the limits defined in the AISC *Code of Standard Practice*. Temporary bracing shall be provided, in accordance with the requirements of the *Code of Standard Practice*, wherever necessary to support all loads to which the structure may be subjected, including equipment and the operation of same. Such bracing shall be left in place as long as required for safety.

13.4.3 **Alignment.** No permanent bolting or welding shall be performed until the adjacent affected portions of the structure have been properly aligned.

13.4.4 **Fit of Column Compression Joints and Base Plates.** Lack of contact bearing not exceeding a gap of 2 mm, regardless of the type of splice used (partial-joint-penetration groove welded, or bolted), is permitted. If the gap exceeds 2 mm, but is less than 6 mm, and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with non-tapered steel shims. Shims need not be other than mild steel, regardless of the grade of the main material.

13.4.5 **Field Welding.** Shop paint on surfaces adjacent to joints to be field welded shall be wire brushed if necessary to assure weld quality.

Field welding of attachments to installed embedments in contact with concrete shall be done in such a manner as to avoid excessive thermal expansion of the embedment which could result in spalling or cracking of the concrete or excessive stress in the embedment anchors.

13.4.6 **Field Painting.** Responsibility for touch-up painting, cleaning, and field painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the contract documents.

13.4.7 **Field Connections.** As erection progresses, the structure shall be securely bolted or welded to support all dead, wind, and erection loads.

SECTION 13.5
QUALITY CONTROL

The fabricator shall provide quality control procedures to the extent that the fabricator deems necessary to assure that all work is performed in accordance with this SBC 306. In addition to the fabricator's quality control procedures, material and workmanship at all times may be subject to inspection by qualified inspectors representing the purchaser. If such inspection by representatives of the purchaser will be required, it shall be so stated in the contract documents.

13.5.1 **Cooperation.** As far as possible, all inspection by representatives of the purchaser shall be made at the fabricator's plant. The fabricator shall cooperate with the inspector, permitting access for inspection to all places where work is being done. The purchaser's inspector shall schedule this work for minimum interruption to the work of the fabricator.

13.5.2 **Rejections.** Material or workmanship not in reasonable conformance with the provisions of SBC 306 may be rejected at any time during the progress of the work. The fabricator shall receive copies of all reports furnished to the purchaser by the inspection agency.

13.5.3 **Inspection of Welding.** The inspection of welding shall be performed in accordance with the provisions of AWS D1.1 except as modified in Section 10.2.
When visual inspection is required to be performed by AWS certified welding inspectors, it shall be so specified in the design documents.
When nondestructive testing is required, the process, extent, and standards of acceptance shall be clearly defined in the contract documents.

13.5.4 **Inspection of Slip-Critical High-Strength Bolted Connections.** The inspection of slip-critical high-strength bolted connections shall be in accordance with the provisions of the RCSC *Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts*.

13.5.5 **Identification of Steel.** The fabricator shall be able to demonstrate by a written procedure and by actual practice a method of material application and identification, visible at least through the "fit-up" operation, of the main structural elements of a shipping piece.

The identification method shall be capable of verifying proper material application as it relates to:

(1) Material specification designation

(2) Heat number, if required

(3) Material test reports for special requirements

CHAPTER 14
EVALUATION OF EXISTING STRUCTURES

This chapter applies to the evaluation of the strength and stiffness under static vertical (gravity) loads of existing structures by structural analysis, by load tests, or by a combination of structural analysis and load tests when specified by the Engineer of Record or in the contract documents. For such evaluation, the steel grades are not limited to those listed in Section 1.3.1. This chapter does not address load testing for the effects of seismic loads or moving loads (vibrations).

SECTION 14.1
GENERAL PROVISIONS

These provisions shall be applicable when the evaluation of an existing steel structure is specified for (a) verification of a specific set of design loadings or (b) determination of the design strength of a load resisting member or system. The evaluation shall be performed by structural analysis (Section 14.3), by load tests (Section 14.4), or by a combination of structural analysis and load tests, as specified in the contract documents. Where load tests are used, the Engineer of Record shall first analyze the structure, prepare a testing plan, and develop a written procedure to prevent excessive permanent deformation or catastrophic collapse during testing.

SECTION 14.2
MATERIAL PROPERTIES

14.2.1 **Determination of Required Tests.** The Engineer of Record shall determine the specific tests that are required from Section 14.2.2 through 14.2.6 and specify the locations where they are required. Where available, the use of applicable project records shall be permitted to reduce or eliminate the need for testing.

14.2.2 **Tensile Properties.** Tensile properties of members shall be considered in evaluation by structural analysis (Section 14.3) or load tests (Section 14.4). Such properties shall include the yield stress, tensile strength, and percent elongation. Where available, certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6 or A568, as applicable, shall be permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with ASTM A370 from samples cut from components of the structure.

14.2.3 **Chemical Composition.** Where welding is anticipated for repair or modification of existing structures, the chemical composition of the steel shall be determined for use in preparing a welding procedure specification (WPS). Where available, results from certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM procedures or Saudi equivalents where applicable shall be permitted for this purpose. Otherwise, analyses shall be conducted in accordance with ASTM A751 from the samples used to determine tensile properties, or from samples taken from the same locations.

14.2.4 **Base Metal Notch Toughness.** Where welded tension splices in heavy shapes and plates as defined in Section 1.3.1.3 are critical to the performance of the structure, the Charpy V-notch toughness shall be determined in accordance with the

provisions of Section 1.3.1.3. If the notch toughness so determined does not meet the provisions of Section 1.3.1.3, the Engineer of Record shall determine if remedial actions are required.

14.2.5 **Weld Metal.** Where structural performance is dependent on existing welded connections, representative samples of weld metal shall be obtained. Chemical analysis and mechanical tests shall be made to characterize the weld metal. A determination shall be made of the magnitude and consequences of imperfections. If the requirements of Saudi equivalent to AWS D1.1 are not met, the Engineer of Record shall determine if remedial actions are required.

14.2.6 **Bolts and Rivets.** Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples shall be removed and tested to determine tensile strength in accordance with ASTM F606 and the bolt classified accordingly. Alternatively, the assumption that the bolts are A307 shall be permitted. Rivets shall be assumed to be A502, Grade 1, unless a higher grade is established through documentation or testing.

SECTION 14.3
EVALUATION BY STRUCTURAL ANALYSIS

14.3.1 **Dimensional Data.** All dimensions used in the evaluation, such as spans, column heights, member spacings, bracing locations, cross section dimensions, thicknesses, and connection details, shall be determined from a field survey. Alternatively, when available, it shall be permitted to determine such dimensions from applicable project design or shop drawings with field verification of critical values.

14.3.2 **Strength Evaluation.** Forces (load effects) in members and connections shall be determined by structural analysis applicable to the type of structure evaluated. The load effects shall be determined for the loads and factored load combinations stipulated in Section 1.4.

The design strength of members and connections shall be determined from applicable provisions of Chapters 2 through 11 of SBC 306.

14.3.3 **Serviceability Evaluation.** Where required, the deformations at service loads shall be calculated and reported.

SECTION 14.4
EVALUATION BY LOAD TESTS

14.4.1 **Determination of Live Load Rating by Testing.** To determine the live load rating of an existing floor or roof structure by testing, test load shall be applied incrementally in accordance with the Engineer of Record's plan. The structure shall be visually inspected for signs of distress or imminent failure at each load level. Appropriate measures shall be taken if these or any other unusual conditions are encountered.

The tested design strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested design strength equal to $1.2D + 1.6L$, where D is

the nominal dead load and L is the nominal live load rating for the structure. The nominal live load rating of the floor structure shall not exceed that which can be calculated using applicable provisions of SBC 306. For roof structures, L_r, or R as defined in the Symbols, shall be substituted for L. More severe load combinations shall be used where required by applicable building codes.

Periodic unloading shall be considered once the service load level is attained and after the onset of inelastic structural behavior is identified to document the amount of permanent set and the magnitude of the inelastic deformations. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining maximum test load for one hour, that the deformation of the structure does not increase by more than 10 percent above that at the beginning of the holding period. It is permissible to repeat the sequence if necessary to demonstrate compliance.

Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Because the amount of acceptable permanent deformation depends on the specific structure, no limit is specified for permanent deformation at maximum loading. Where it is not feasible to load test the entire structure, a segment or zone of not less than one complete bay, representative of the most critical conditions, shall be selected.

14.4.2 **Serviceability Evaluation.** When load tests are prescribed, the structure shall be loaded incrementally to the service load level. Deformations shall be monitored for a period of one hour. The structure shall then be unloaded and the deformation recorded.

SECTION 14.5
EVALUATION REPORT

After the evaluation of an existing structure has been completed, the Engineer of Record shall prepare a report documenting the evaluation. The report shall indicate whether the evaluation was performed by structural analysis, by load testing or by a combination of structural analysis and load testing. Furthermore, when testing is performed, the report shall include the loads and load combination used and the load-deformation and time-deformation relationships observed. All relevant information obtained from design drawing, mill test reports, and auxiliary material testing shall also be reported. Finally, the report shall indicate whether the design strength of the structure, including all members and connections, is adequate to withstand the load effects.

APPENDIX A:

GLOSSARY

Alignment chart for columns. A nomograph for determining the effective length factor *K* for some types of columns.

Amplification factor. A multiplier of the value of moment or deflection in the unbraced length of an axially loaded member to reflect the secondary values generated by the eccentricity of the applied axial load within the member.

Aspect ratio. In any rectangular configuration, the ratio of the lengths of the sides.

Batten plate. A plate element used to join two parallel components of a built-up column, girder, or strut rigidly connected to the parallel components and designed to transmit shear between them.

Beam. A structural member whose primary function is to carry loads transverse to its longitudinal axis.

Beam-column. A structural member whose primary function is to carry loads both transverse and parallel to its longitudinal axis.

Bent. A plane framework of beam or truss members which support loads and the columns which support these members.

Biaxial bending. Simultaneous bending of a member about two perpendicular axes.

Bifurcation. The phenomenon whereby a perfectly straight member under compression may either assume a deflected position or may remain undeflected, or a beam under flexure may either deflect and twist out of plane or remain in its in-plane deflected position.

Braced frame. A frame in which the resistance to lateral load or frame instability is primarily provided by a diagonal, a *K* brace, or other auxiliary system of bracing.

Brittle fracture. Abrupt cleavage with little or no prior ductile deformation.

Buckling load. The load at which a perfectly straight member under compression assumes a deflected position.

Built-up member. A member made of structural metal elements that are welded, bolted, or riveted together.

Charpy V-notch impact test. A standard dynamic test in which a notched specimen is struck and broken by a single blow in a specially designed testing machine. The measured test values may be the energy absorbed, the percentage shear fracture, the lateral expansion opposite the notch, or a combination thereof.

Cladding. The exterior covering of the structural components of a building.

Cold-formed members. Structural members formed from steel without the application of heat.

Column. A structural member whose primary function is to carry loads parallel to its longitudinal axis.

Column curve. A curve expressing the relationship between axial column strength and slenderness ratio.

Combined mechanism. A mechanism determined by plastic analysis procedure which combines elementary beam, panel, and joint mechanisms.

Compact section. Compact sections are capable of developing a fully plastic stress distribution and possess rotation capacity of approximately three before the onset of local buckling.

Composite beam. A steel beam structurally connected to a concrete slab so that the beam and slab respond to loads as a unit.

Concrete-encased beam. A beam totally encased in concrete cast integrally with the slab.

Connection. Combination of joints used to transmit forces between two or more members. Categorized by the type and amount of force transferred (moment, shear, end reaction). See also *Splices.*

Critical load. The load at which bifurcation occurs as determined by a theoretical stability analysis.

Curvature. Rotation per unit length due to bending.

Design documents. Documents prepared by the designer (design drawings, design details, and job specifications).

Design strength. Resistance (force, moment, stress, as appropriate) provided by element or connection; the product of the nominal strength and the resistance factor.

Diagonal bracing. Inclined structural members carrying primarily axial load enabling a structural frame to act as a truss to resist horizontal loads.

Diaphragm. Floor slab, metal wall, or roof panel possessing a large in-plane shear stiffness and strength adequate to transmit horizontal forces to resisting systems.

Diaphragm action. The in-plane action of a floor system (also roofs and walls) such that all columns framing into the floor from above and below are maintained in the same position relative to each other.

Double concentrated forces. Two equal and opposite forces which form a couple on the same side of the loaded member.

Double curvature. A bending condition in which end moments on a member cause the member to assume an S shape.

Drift. Lateral deflection of a building.

Drift index. The ratio of lateral deflection to the height of the building.

Ductility factor. The ratio of the total deformation at maximum load to the elastic-limit deformation.

Effective length. The equivalent length KL used in compression formulas and determined by a bifurcation analysis.

Effective length factor K. The ratio between the effective length and the unbraced length of the member measured between the centers of gravity of the bracing members.

Effective moment of inertia. The moment of inertia of the cross section of a member that remains elastic when partial plastification of the cross section takes place, usually under the combination of residual stress and applied stress. Also, the moment of inertia based on effective widths of elements that buckle locally. Also, the moment of inertia used in the

design of partially composite members.

Effective stiffness. The stiffness of a member computed using the effective moment of inertia of its cross section.

Effective width. The reduced width of a plate or slab which, with an assumed uniform stress distribution, produces the same effect on the behavior of a structural member as the actual plate width with its non uniform stress distribution.

Elastic analysis. Determination of load effects (force, moment, stress, as appropriate) on members and connections based on the assumption that material deformation disappears on removal of the force that produced it.

Elastic-perfectly plastic. A material which has an idealized stress-strain curve that varies linearly from the point of zero strain and zero stress up to the yield point of the material, and then increases in strain at the value of the yield stress without any further increases in stress.

Embedment. A steel component cast in a concrete structure which is used to transmit externally applied loads to the concrete structure by means of bearing, shear, bond, friction, or any combination thereof. The embedment may be fabricated of structural-steel plates, shapes, bars, bolts, pipe, studs, concrete reinforcing bars, shear connectors, or any combination thereof.

Encased steel structure. A steel-framed structure in which all of the individual frame members are completely encased in cast-in-place concrete.

Euler formula. The mathematical relationship expressing the value of the Euler load in terms of the modulus of elasticity, the moment of inertia of the cross section, and the length of a column.

Euler load. The critical load of a perfectly straight, centrally loaded pin-ended column.

Eyebar. A particular type of pin-connected tension member of uniform thickness with forged or flame-cut head of greater width than the body proportioned to provide approximately equal strength in the head and body.

Factored load. The product of the nominal load and a load factor *Fastener.* Generic term for welds, bolts, rivets, or other connecting device *Fatigue.* A fracture phenomenon resulting from a fluctuating stress cycle.

First-order analysis. Analysis based on first-order deformations in which equilibrium conditions are formulated on the undeformed structure.

Flame-cut plate. A plate in which the longitudinal edges have been prepared by oxygen cutting from a larger plate.

Flat width. For a rectangular HSS, the nominal width minus twice the outside corner radius. In absence of knowledge of the corner radius, the flat width may be taken as the total section width minus three times the thickness.

Flexible connection. A connection permitting a portion, but not all, of the simple beam rotation of a member end.

Floor system. The system of structural components separating the stories of a building.

Force. Resultant of distribution of stress over a prescribed area. A reaction that develops in a member as a result of load (formerly called total stress or stress). Generic term signifying

axial loads, bending moment, torques, and shears.

Fracture toughness. Measure of the ability to absorb energy without fracture. Generally determined by impact loading of specimens containing a notch having a prescribed geometry.

Frame buckling. A condition under which bifurcation may occur in a frame.

Frame instability. A condition under which a frame deforms with increasing lateral deflection under a system of increasing applied monotonic loads until a maximum value of the load called the stability limit is reached, after which the frame will continue to deflect without further increase in load.

Fully composite beam. A composite beam with sufficient shear connectors to develop the full flexural strength of the composite section.

High-cycle fatigue. Failure resulting from more than 20,000 applications of cyclic stress.

HSS. Hollow structural sections that are prismatic square, rectangular or round products of a pipe or tubing mill and meet the geometric tolerance, tensile strength and chemical composition requirements of a standard specification.

Hybrid beam. A fabricated steel beam composed of flanges with a greater yield strength than that of the web. Whenever the maximum flange stress is less than or equal to the web yield stress the girder is considered homogeneous.

Hysteresis loop. A plot of force versus displacement of a structure or member subjected to reversed, repeated load into the inelastic range, in which the path followed during release and removal of load is different from the path for the addition of load over the same range of displacement.

Inclusions. Nonmetallic material entrapped in otherwise sound metal.

Incomplete fusion. Lack of union by melting of filler and base metal over entire prescribed area.

Inelastic action. Material deformation that does not disappear on removal of the force that produced it.

Instability. A condition reached in the loading of an element or structure in which continued deformation results in a decrease of load-resisting capacity.

Joint. Area where two or more ends, surfaces, or edges are attached. Categorized by type of fastener or weld used and method of force transfer.

K bracing. A system of struts used in a braced frame in which the pattern of the struts resembles the letter K, either normal or on its side.

Lamellar tearing. Separation in highly restrained base metal caused by through-thickness strains induced by shrinkage of adjacent filler metal.

Lateral bracing member. A member utilized individually or as a component of a lateral bracing system to prevent buckling of members or elements and/or to resist lateral loads.

Lateral (or lateral-torsional) buckling. Buckling of a member involving lateral deflection and twist.

Leaning column. Gravity-loaded column where connections to the frame (simple connections) do not provide resistance to lateral loads.

Limit state. A condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (*serviceability limit state)* or to be unsafe (*strength limit state).*

Limit states. Limits of structural usefulness, such as brittle fracture, plastic collapse, excessive deformation, durability, fatigue, instability, and serviceability.

Load factor. A factor that accounts for unavoidable deviations of the actual load from the nominal value and for uncertainties in the analysis that transforms the load into a load effect.

Loads. Forces or other actions that arise on structural systems from the weight of all permanent construction, occupants and their possessions, environmental effects, differential settlement, and restrained dimensional changes. *Permanent* loads are those loads in which variations in time are rare or of small magnitude. All other loads are *variable* loads. See *Nominal loads.*

LRFD (Load and Resistance Factor Design). A method of proportioning structural components (members, connectors, connecting elements, and assemblages) such that no applicable limit state is exceeded when the structure is subjected to all appropriate load combinations.

Local buckling. The buckling of a compression element which may precipitate the failure of the whole member.

Low-cycle fatigue. Fracture resulting from a relatively high-stress range resulting in a relatively small number of cycles to failure.

Lower bound load. A load computed on the basis of an assumed equilibrium moment diagram in which the moments are not greater than M_p that is less than or at best equal to the true ultimate load.

Mechanism. An articulated system able to deform without an increase in load, used in the special sense that the linkage may include real hinges or plastic hinges, or both.

Mechanism method. A method of plastic analysis in which equilibrium between external forces and internal plastic hinges is calculated on the basis of an assumed mechanism. The failure load so determined is an upper bound.

Nodal Brace. A brace that prevents the lateral movement or twist at the particular brace location along the length of the beam or column without any direct attachment to other braces at adjacent brace points. (See relative brace).

Nominal loads. The magnitudes of the loads specified by the applicable code.

Nominal strength. The capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Noncompact section. Noncompact sections can develop the yield stress in compression elements before local buckling occurs, but will not resist inelastic local buckling at strain levels required for a fully plastic stress distribution.

P-Delta effect. Secondary effect of column axial loads and lateral deflection on the moments in members.

Panel zone. The zone in a beam-to-column connection that transmits moment by a shear panel.

Partially composite beam. A composite beam for which the shear strength of shear connectors governs the flexural strength.

Plane frame. A structural system assumed for the purpose of analysis and design to be two-dimensional.

Plastic analysis. Determination of load effects (force, moment, stress, as appropriate) on members and connections based on the assumption of rigid-plastic behavior, i.e., that equilibrium is satisfied throughout the structure and yield is not exceeded anywhere. Second order effects may need to be considered.

Plastic design section. The cross section of a member which can maintain a full plastic moment through large rotations so that a mechanism can develop; the section suitable for design by plastic analysis.

Plastic hinge. A yielded zone which forms in a structural member when the plastic moment is attained. The beam is assumed to rotate as if hinged, except that it is restrained by the plastic moment M_p.

Plastic-limit load. The maximum load that is attained when a sufficient number of yield zones have formed to permit the structure to deform plastically without further increase in load. It is the largest load a structure will support, when perfect plasticity is assumed and when such factors as instability, second-order effects, strain hardening, and fracture are neglected.

Plastic mechanism. See Mechanism.

Plastic modulus. The section modulus of resistance to bending of a completely yielded cross section. It is the combined static moment about the plastic neutral axis of the cross-sectional areas above and below that axis.

Plastic moment. The resisting moment of a fully-yielded cross section.

Plastic strain. The difference between total strain and elastic strain *Plastic zone.* The yielded region of a member.

Plastification. The process of successive yielding of fibers in the cross section of a member as bending moment is increased.

Plate girder. A built-up structural beam.

Post-buckling strength. The load that can be carried by an element, member, or frame after buckling.

Prying Action. Lever action that exists in connections in which the line of application of the applied load is eccentric to the axis of the bolt, causing deformation of the fitting and an amplification of the axial force in the bolt.

Redistribution of moment. A process which results in the successive formation of plastic hinges so that less highly stressed portions of a structure may carry increased moments.

Relative Brace. A brace that controls the relative movement of two adjacent brace points along the length of a beam or column or the relative lateral displacement of two stories in a frame. (See nodal brace).

Required strength. Load effect (force, moment, stress, as appropriate) acting on element or connection determined either by structural analysis from the factored loads (using appropriate critical load combinations) or explicitly specified.

Residual stress. The stresses that remain in an unloaded member after it has been formed into a finished product. (Examples of such stresses include, but are not limited to, those induced by cold bending, cooling after rolling, or welding).

Resistance. The capacity of a structure or component to resist the effects of loads. It is determined by computations using specified material strengths, dimensions and formulas derived from accepted principles of structural mechanics, or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions. Resistance is a generic term that includes both strength and serviceability limit states.

Resistance factor. A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure.

Rigid frame. A structure in which connections maintain the angular relationship between beam and column members under load.

Root of the flange. Location on the web of the corner radius termination point to the toe of the flange-to-web weld. Measured as the k distance from the far side of the flange.

Rotation capacity. The incremental angular rotation that a given shape can accept prior to local failure defined as $R = (O_u / O_p)$ where O_u is the overall rotation attained at the factored load state and Op is the idealized rotation corresponding to elastic theory applied to the case of $M = M_p$.

St. Venant torsion. That portion of the torsion in a member that induces only shear stresses in the member.

Second-order analysis. Analysis based on second-order deformations, in which equilibrium conditions are formulated on the deformed structure.

Service load. Load expected to be supported by the structure under normal usage; often taken as the nominal load.

Serviceability limit state. Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability, or the comfort of its occupants or function of machinery under normal usage.

Shape factor. The ratio of the plastic moment to the yield moment, or the ratio of the plastic modulus to the section modulus for a cross section.

Shear friction. Friction between the embedment and the concrete that transmits shear loads. The relative displacement in the plane of the shear load is considered to be resisted by shear-friction anchors located perpendicular to the plane of the shear load.

Shear lugs. Plates, welded studs, bolts, and other steel shapes that are embedded in the concrete and located transverse to the direction of the shear force and that transmit shear loads, introduced into the concrete by local bearing at the shear lug-concrete interface.

Shear wall. A wall that resists, in its own plane, shear forces resulting from applied wind, earthquake, or other transverse loads or provides frame stability. Also called a structural wall.

Sidesway. The lateral movement of a structure under the action of lateral loads, unsymmetrical vertical loads, or unsymmetrical properties of the structure.

Sidesway buckling. The buckling mode of a multistory frame precipitated by the relative lateral displacements of joints, leading to failure by sidesway of the frame.

Simple plastic theory. See Plastic design.

Single curvature. A deformed shape of a member having one smooth continuous arc, as opposed to double curvature which contains a reversal.

Slender-element section. The cross section of a member which will experience local buckling in the elastic range.

Slenderness ratio. The ratio of the effective length of a column to the radius of gyration of the column, both with respect to the same axis of bending.

Slip-critical joint. A bolted joint in which the slip resistance of the connection is required.

Spaceframe. A three-dimensional structural framework (as contrasted to a plane frame).

Splice. The connection between two structural elements joined at their ends to form a single, longer element.

Stability-limit load. Maximum (theoretical) load a structure can support when second-order instability effects are included.

Stepped column. A column with changes from one cross section to another occurring at abrupt points within the length of the column.

Stiffener. A member, usually an angle or plate, attached to a plate or web of a beam or girder to distribute load, to transfer shear, or to prevent buckling of the member to which it is attached.

Stiffness. The resistance to deformation of a member or structure measured by the ratio of the applied force to the corresponding displacement.

Story drift. The difference in horizontal deflection at the top and bottom of a story.

Strain hardening. Phenomenon wherein ductile steel, after undergoing considerable deformation at or just above yield point, exhibits the capacity to resist substantially higher loading than that which caused initial yielding.

Strain-hardening strain. For structural steels that have a flat (plastic) region in the stress-strain relationship, the value of the strain at the onset of strain hardening.

Strength design. A method of proportioning structural members using load factors and resistance factors such that no applicable limit state is exceeded (also called load and resistance factor design).

Strength limit state. Limiting condition affecting the safety of the structure, in which the ultimate load-carrying capacity is reached.

Stress. Force per unit area.

Stress concentration. Localized stress considerably higher than average (even in uniformly loaded cross sections of uniform thickness) due to abrupt changes in geometry or localized loading.

Strong axis. The major principal axis of a cross section.

Structural system. An assemblage of load-carrying components which are joined together to provide regular interaction or interdependence.

Stub column. A short compression-test specimen, long enough for use in measuring the stress-strain relationship for the complete cross section, but short enough to avoid buckling as a column in the elastic and plastic ranges.

Sub assemblage. A truncated portion of a structural frame.

Supported frame. A frame which depends upon adjacent braced or unbraced frames for resistance to lateral load or frame instability. (This transfer of load is frequently provided by the floor or roof system through diaphragm action or by horizontal cross bracing in the roof).

Tangent modulus. At any given stress level, the slope of the stress-strain curve of a material in the inelastic range as determined by the compression test of a small specimen under controlled conditions.

Temporary structure. A general term for anything that is built or constructed (usually to carry construction loads) that will eventually be removed before or after completion of construction and does not become part of the permanent structural system.

Tensile strength. The maximum tensile stress that a material is capable of sustaining.

Tension field action. The behavior of a plate girder panel under shear force in which diagonal tensile stresses develop in the web and compressive forces develop in the transverse stiffeners in a manner analogous to a Pratt truss.

Toe of the fillet. Termination point of fillet weld or of rolled section fillet.

Torque-tension relationship. Term applied to the wrench torque required to produce specified pre-tension in high-strength bolts.

Turn-of-nut method. Procedure whereby the specified pre-tension in high-strength bolts is controlled by rotation of the wrench a predetermined amount after the nut has been tightened to a snug fit.

Unbraced frame. A frame in which the resistance to lateral load is provided by the bending resistance of frame members and their connections.

Unbraced length. The distance between braced points of a member, measured between the centers of gravity of the bracing members.

Undercut. A notch resulting from the melting and removal of base metal at the edge of a weld.

Universal-mill plate. A plate in which the longitudinal edges have been formed by a rolling process during manufacture.

Often abbreviated as UM plate.

Upper bound load. A load computed on the basis of an assumed mechanism which will always be at best equal to or greater than the true ultimate load.

Vertical bracing system. A system of shear walls, braced frames, or both, extending through one or more floors of a building.

Von Mises yield criterion. A theory which states that inelastic action at any point in a body under any combination of stresses begins only when the strain energy of distortion per unit volume absorbed at the point is equal to the strain energy of distortion absorbed per unit volume at any point in a simple tensile bar stressed to the elastic limit under a state of uniaxial stress. It is often called the maximum strain-energy-of-distortion theory. Accordingly, shear yield occurs at 0.58 times the yield strength.

Warping torsion. That portion of the total resistance to torsion that is provided by resistance to warping of the cross section.

Weak axis. The minor principal axis of a cross section.

Weathering steel. A type of high-strength, low-alloy steel which can be used in normal environments (not marine) and outdoor exposures without protective paint covering. This steel develops tight adherent rust at a decreasing rate with respect to time.

Web buckling. The buckling of a web plate.

Web crippling. The local failure of a web plate in the immediate vicinity of a concentrated load or reaction.

Working load. Also called service load. The actual load assumed to be acting on the structure.

Yield moment. In a member subjected to bending, the moment at which an outer fiber first attains the yield stress.

Yield plateau. The portion of the stress-strain curve for uniaxial tension or compression in which the stress remains essentially constant during a period of substantially increased strain.

Yield point. The first stress in a material at which an increase in strain occurs without an increase in stress.

Yield strength. The stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain. Deviation expressed in terms of strain.

Yield stress. Yield point, yield strength, or yield stress level as defined.

Yield-stress level. The average stress during yielding in the plastic range, the stress determined in a tension test when the strain reaches 0.005 mm per mm.

APPENDIX B:

SYMBOLS

The section number in the right hand column refers to the section where the symbol is first used.

Symbol	Definition	Section
A	Area of directly connected elements	2.3
A_B	Loaded area of concrete, mm^2	9.2.4
Ab	Nominal unthreaded body area of bolt or threaded part, mm^2	10.3.6
A_c	Area of concrete, mm^2	9.2.2
A_c	Area of concrete slab within effective width, mm^2	9.5.2
A_D	Area of an upset rod based on the major thread diameter, mm^2	10.3.6
A_e	Effective area, mm^2	2.3
A_f	Area of the compression flange, mm^2	6.3.4
A_{fe}	Effective tension flange area, mm^2	2.10
A_{fg}	Gross area of flange, mm^2	2.10
A_{fn}	Net area of flange, mm^2	2.10
A_g	Gross area, mm^2	1.5
A_{gt}	Gross area subject to tension, mm^2	10.4.3
A_{gv}	Gross area subject to shear, mm^2	10.4.3
A_n	Net area, mm^2	2.3
A_{nt}	Net area subject to tension, mm^2	10.4.2
A_{nv}	Net area subject to shear, mm^2	10.4.1
A_{pb}	Projected bearing area, mm^2	10.8
A_r	Area of reinforcing bars, mm^2	9.2.2
A_s	Area of steel cross section, mm^2	9.2.2
A_{sc}	Cross-sectional area of stud shear connector, mm^2	9.5.3
A_{sf}	Shear area on the failure path, mm^2	4.3
A_{st}	Area of a transverse stiffener, mm^2	7.4
A_t	Net tensile area, mm^2	11.3.5
A_w	Web area, mm^2	6.2.1
A_1	Area of steel concentrically bearing on a concrete support, m^2	10.9
A_2	Total cross-sectional area of a concrete support, mm^2	10.9
B	Factor for bending stress in tees and double angles	6.1.2
B	Factor for bending stress in web-tapered members, mm, defined by Equations 6.3-8 through 6.3-11	6.3
B_1, B_2	Factors used in determining M_u for combined bending and axial forces when first-order analysis is employed	3.1
C_{PG}	Plate-girder coefficient	7.2
C_b	Bending coefficient dependent on moment gradient	6.1.2
C_f	Constant based on stress category, given in Table 11.3.1	11.3.3
C_m	Coefficient applied to bending term in interaction formula for prismatic members and dependent on column curvature caused by applied moments	3.1
$C_m^{'}$	Coefficient applied to bending term in interaction formula for tapered members and dependent on axial stress at the small end of the member	6.3.6

C_p	Ponding flexibility coefficient for primary member in a flat roof	11.2
C_s	Ponding flexibility coefficient for secondary member in a flat roof	11.2
C_v	Ratio of "critical" web stress, according to linear buckling theory, to the shear yield stress of web material	7.3
C_w	Warping constant, mm^6	6.1.2
D	Outside diameter of circular hollow section, mm	2.5.3
D	Factor used in Equation 7.4-1, dependent on the type of transverse stiffeners used in a plate girder	7.4
E	Modulus of elasticity of steel, E = 200,000 MPa	5.2
E_c	Modulus of elasticity of concrete, MPa	9.2.2
E_m	Modified modulus of elasticity, MPa	9.2.2
F_{BM}	Nominal strength of the base material to be welded, MPa	10.2.4
F_{EXX}	Classification number of weld metal (minimum specified strength), MPa	10.2.4
F_L	Smaller of $(F_{yf}-F_r)$ or F_{yw}, MPa	6.1.2
F_{SR}	Design stress range, MPa	11.3.3
F_{TH}	Threshold fatigue stress range, maximum stress range for indefinite design life, MPa	11.3.3
$F_{b\gamma}$	Flexural stress for tapered members defined by Equations 6.3-4 and 6.3-5	6.3.4
F_{cr}	Critical stress, MPa	5.2
F_{crft}, F_{cry}, F_{crz}	Flexural-torsional buckling stresses for double-angle and tee-shaped compression members, MPa	5.3
F_e	Elastic buckling stress, MPa	5.3
F_{ex}	Elastic flexural buckling stress about the major axis, MPa	5.3
F_{ey}	Elastic flexural buckling stress about the minor axis, MPa	5.3
F_{ez}	Elastic torsional buckling stress, MPa	5.3
F_{my}	Modified yield stress for composite columns, MPa	9.2.2
R_n	Nominal shear rupture strength, MPa	10.4
F_r	Compressive residual stress in flange 69 MPa for rolled shapes; 114 MPa for welded built-up shapes	2.5.1
$F_{s\gamma}$	Stress for tapered members defined by Equation 6.3-6, MPa	6.3.4
F_u	Specified minimum tensile strength of the type of steel being used, MPa	2.10
F_w	Nominal strength of the weld electrode material, MPa	10.2.4
$F_{w\gamma}$	Stress for tapered members defined by Equation 6.3-7, MPa	6.3.4
F_y	Specified minimum yield stress of the type of steel being used, MPa. As used in this Specification, "yield stress" denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point)	1.5
F_{yf}	Specified minimum yield stress of the flange, MPa	2.5.1
F_{yr}	Specified minimum yield stress of reinforcing bars, MPa	9.2.2
F_{yst}	Specified minimum yield stress of the stiffener material, MPa	7.4
F_{yw}	Specified minimum yield stress of the web, MPa	2.5.1
G	Shear modulus of elasticity of steel, G = 77,200 MPa	6.1.2
H	Horizontal force, N	3.1
H	Flexural constant	5.3
H_s	Length of stud connector after welding, mm	9.3.5

I	Moment of inertia, mm^4	6.1.2
I_d	Moment of inertia of the steel deck supported on secondary members, mm^4	11.2
I_p	Moment of inertia of primary members, mm^4	11.2
I_s	Moment of inertia of secondary members, mm^4	11.2
J	Torsional constant for a section, mm^4	6.1.2
K	Effective length factor for prismatic member	2.7
K_z	Effective length factor for torsional buckling	5.3
K_γ	Effective length factor for a tapered member	6.3.3
L	Story height or panel spacing, mm	3.1
L	Live load due to occupancy and moveable equipment	14.4
L_b	Laterally unbraced length; length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section, mm	6.1.2
L_c	Length of channel shear connector, mm	9.5.4
L_c	Edge distance, mm.	10.3.10
L_p	Limiting laterally unbraced length for full plastic bending capacity, uniform moment case (C_b = 1.0), mm	6.1.2
L_p	Column spacing in direction of girder, m	11.2
L_{pd}	Limiting laterally unbraced length for plastic analysis, m	6.1.2
L_q	Maximum unbraced length for the required column force with K equal to one, mm	3.3
L_r	Limiting laterally unbraced length for inelastic lateral-torsional buckling, mm	6.1.2
L_r	Roof live load	14.4
L_s	Column spacing perpendicular to direction of girder, m	11.2
M_A	Absolute value of moment at quarter point of the unbraced beam segment, N-mm	6.1.2
M_B	Absolute value of moment at centerline of the unbraced beam segment, N-mm	6.1.2
M_C	Absolute value of moment at three-quarter point of the unbraced beam segment, N-mm	6.1.2
M_{cr}	Elastic buckling moment, N-mm	6.1.2
M_{lt}	Required flexural strength in member due to lateral frame translation only, N-mm.	3.1
M_{max}	Absolute value of maximum moment in the unbraced beam segment, N-mm	6.1.2
M_n	Nominal flexural strength, N-mm	6.1.1
M'_{nx}, M'_{ny}	Flexural strength defined in Equations 8.3-12 and 8.3-13 for use in alternate interaction equations for combined bending and axial force, N-mm	8.3
M_{nt}	Required flexural strength in member assuming there is no lateral translation of the frame, N-mm	3.1.2
M_p	Plastic bending moment, N-mm	6.1.1
M_p	Moment defined in Equations 8.3-11(a) and 8.3-11(b), for use in alternate interaction equations for combined bending and axial force, N-mm	8.3
M_r	Limiting buckling moment, $M_{cr,}$ when $\lambda = \lambda_r$ and C_b = 1.0, N-mm	6.1.2
M_u	Required flexural strength, kip-in. N-mm	3.1

M_y	Moment corresponding to onset of yielding at the extreme fiber from an elastic stress distribution (= F_y S for homogeneous sections), N-mm	6.1.1
M_1	Smaller moment at end of unbraced length of beam or beam-column, N-mm	6.1.3
M_2	Larger moment at end of unbraced length of beam or beam-column, N-mm	6.1.3
N	Length of bearing, mm	11.1.3
N	Number of stress range fluctuations in design life	11.3.3
N_r	Number of stud connectors in one rib at a beam intersection	9.3.5
P_{br}	Required story or panel bracing shear force	3.3
P_{e1}, P_{e2}	Elastic Euler buckling load for braced and unbraced frame, respectively, N	3.1
P_n	Nominal axial strength (tension or compression), N	4.1
P_p	Bearing load on concrete, N	10.9
P_u	Required axial strength (tension or compression),	2.5.1
P_y	Yield strength,	2.5.1
Q	Full reduction factor for slender compression elements	5.3
Q_a	Reduction factor for slender stiffened compression elements	2.5
Q_n	Nominal strength of one stud shear connector,	9.5
Q_s	Reduction factor for slender unstiffened compression elements	2.5.3
R	Nominal load due to initial rainwater or ice exclusive of the ponding contribution	14.4
R_{PG}	Plate girder bending strength reduction factor	7.2
R_e	Hybrid girder factor	7.2
R_n	Nominal strength	1.5.3
R_v	Web shear strength, N	11.1.7
S	Elastic section modulus, mm^3	6.1.2
S	Spacing of secondary members, m	11.2
S'_x	Elastic section modulus of larger end of tapered member about its major axis, mm^3	6.3
T	Tension force due to service loads, N	10.3.9
T_b	Specified pretension load in high-strength bolt,	10.3.9
T_u	Required tensile strength due to factored loads,	10.3.9
U	Reduction coefficient, used in calculating effective net area	2.3
V_n	Nominal shear strength, N	6.2.2
V_u	Required shear strength,	7.4
W	Wind load	C1.4
X_1	Beam buckling factor defined by Equation 6.1-8	6.1.2
X_2	Beam buckling factor defined by Equation 6.1-9	6.1.2
Z	Plastic section modulus, mm^3	6.1.1
a	Clear distance between transverse stiffeners, mm	6.2.2
a	Distance between connectors in a built-up member, mm	5.4
a	Shortest distance from edge of pin hole to edge of member measured parallel to direction of force, mm	4.3
a_r	Ratio of web area to compression flange area	7.2
a'	Weld length, mm	2.10
b	Compression element width, mm	2.5.1
b_e	Reduced effective width for slender compression elements, mm	2.5.3
b_{eff}	Effective edge distance, mm	4.3
b_f	Flange width, mm	2.5.1
b_s	Stiffener width for one-sided stiffeners, mm	3.3.4

c_1, c_2, c_3	Numerical coefficients	9.2.2
d	Nominal fastener diameter, mm	10.3.3
d	Overall depth of member, mm	2.5.1
d	Pin diameter, mm	4.3
d	Roller diameter, mm	10.8
d_L	Depth at larger end of unbraced tapered segment, mm	6.3
d_b	Beam depth, mm.	11.1.7
d_b	Nominal diameter (body or shank diameter), mm	11.3.3
d_c	Column depth, mm.	11.1.7
d_o	Depth at smaller end of unbraced tapered segment, mm	6.3
e	Base of natural logarithm = 2.71828.	C5.2
f	Computed compressive stress in the stiffened element, MPa	2.5.3
f_{b1}	Smallest computed bending stress at one end of a tapered segment, MPa	6.3
f_{b2}	Largest computed bending stress at one end of a tapered segment, MPa	6.3
$f_c^{'}$	Specified compressive strength of concrete, MPa.	10.2.2
f_o	Stress due to 1.2D + 1.2R, MPa	11.3
f_{un}	Required normal stress, MPa	8.2
f_{uv}	Required shear stress, MPa	8.2
f_v	Required shear stress due to factored loads in bolts or rivets, MPa	10.3.7
g	Transverse center-to-center spacing (gage) between fastener gage lines, mm	2.2
h	Clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, mm	2.5.1
h	Distance between centroids of individual components perpendicular to the member axis of buckling, mm.	5.4.1
h_c	Twice the distance from the centroid to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used, for built-up sections, mm	2.5.1
h_o	Distance between flange centroids, mm	3.3.4.1
h_r	Nominal rib height, mm	9.3.5.2
h_s	Factor used in Equation 6.3-6 for web-tapered members	6.3.4
h_w	Factor used in Equation 6.3-7 for web-tapered members	6.3.4
j	Factor defined by Equation 6.2-4 for minimum moment of inertia for a transverse stiffener	6.2.3
k	Distance from outer face of flange to web toe of fillet, mm	11.1.3
k_v	Web plate buckling coefficient	6.2.2
l	Laterally unbraced length of member at the point of load, mm	2.7
l	Length of bearing, mm	10.8
l	Length of connection in the direction of loading, mm	2.3
l	Length of weld, mm	2.3
l	Length of connection in the direction of loading, mm.	2.3
m	Ratio of web to flange yield stress or critical stress in hybrid beams	7.2
n	Number of nodal braced points within the span	3.3.4.2
r	Governing radius of gyration, mm	2.7

r_{To}	For the smaller end of a tapered member, the radius of gyration, considering only the compression flange plus one-third of the compression web area, taken about an axis in the plane of the web, mm	6.3.4
r_i	Minimum radius of gyration of individual component in a built-up member, mm	5.4.1
r_{ib}	Radius of gyration of individual component relative to centroidal axis parallel to member axis of buckling, mm.	5.4.1
r_m	Radius of gyration of the steel shape, pipe, or tubing in composite columns. For steel shapes it may not be less than 0.3 times the overall thickness of the composite section, mm	9.2.2
r_o	Polar radius of gyration about the shear center, mm	5.3.1
r_{ox}, r_{oy}	Radius of gyration about x and y axes at the smaller end of a tapered member, respectively, mm	6.3.3
r_x, r_y	Radius of gyration about x and y axes, respectively, mm	5.3.2
r_{yc}	Radius of gyration about y axis referred to compression flange, or if reverse curvature bending, referred to smaller flange, mm	6.1
s	Longitudinal center-to-center spacing (pitch) of any two consecutive holes, mm	2.2
t	Thickness of element, mm	2.5.1
t	HSS design wall thickness, mm	2.5.1
t_f	Flange thickness of channel shear connector, mm	9.5.4
t_s	Web stiffener thickness, mm	3.3.4.2
t_w	Web thickness of channel shear connector, mm	9.5.4
t_w	Web thickness, mm	2.5.3.1
w	Leg size of the fillet weld, mm	10.2.2
w	Plate width; distance between welds, mm	2.3
w	Unit weight of concrete, kg/m^3	9.2.2
w_r	Average width of concrete rib or haunch, mm	9.3.5.2
x_o, y_o	Coordinates of the shear center with respect to the centroid, m	5.3
x	Connection eccentricity, mm	2.3
z	Distance from the smaller end of tapered member used in Equation 6.3-1 for the variation in depth, mm.	6.3
α	Separation ratio for built-up compression members = $h / 2r_{ib.}$	5.4.1
β	Reduction factor given by Equation 10.2-1	10.2.1
β_T	Brace stiffness requirement when there is no web Distortion	3.3.4.2
β_{Tb}	Required nodal torsional bracing stiffness	3.3.4.2
β_{br}	Required story or panel shear stiffness	3.3
$\beta_{\sec}$	Web distortional stiffness, including the effect of web transverse stiffeners, if any	3.3.4.2
Δ_{oh}	Translation deflection of the story under consideration, m	3.1.2
γ	Depth tapering ratio	6.3
γ	Subscript for tapered members	6.3
ζ	Exponent for alternate beam-column interaction equation	8.3
η	Exponent for alternate beam-column interaction equation	8.3
λ_c	Column slenderness parameter	5.2
λ_e	Equivalent slenderness parameter	5.3.2
λ_{eff}	Effective slenderness ratio defined by Equation 6.3-2	6.3
λ_p	Limiting slenderness parameter for compact element	2.5.1

λ_r	Limiting slenderness parameter for noncompact element	2.5.1
ϕ	Resistance factor	1.5.3
ϕ_b	Resistance factor for flexure	6.1.1
ϕ_c	Resistance factor for compression	1.5.1
ϕ_c	Resistance factor for axially loaded composite columns	9.2.2
ϕ_t	Resistance factor for tension	4.1
ϕ_y	Resistance factor for shear	6.2.2